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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PRECISE WEIR MEASUREMENTS

By Ernest W. Schoder,* M. Am. Soc. C. E., and the Late Kenneth B. Turner,† Esq.

Synopsis

This paper presents the results of extensive new volumetric measurements of the discharge over weirs of the sharp-crested (or nearly sharp) type, occupying the full width of the channel (that is, without end contractions). In addition to experimental data by the writers, the paper includes also new experimental data by Charles L. Walker, M. Am. Soc. C. E., and Carl R. Weidner, Assoc M. Am. Soc. C. E., William F. Martin, Francis M. Dawson, M. Am. Soc. C. E., Henry R. J. Meyer, Assoc. M. Am. Soc. C. E., and Russell A. See, Jacob O. Jones, Assoc. M. Am. Soc. C. E., and Charles M. Weber, Assoc. M. Am. Soc. C. E.

General.—The heights of the weirs ranged from 0.5 to 7.5 ft., the heads, from 0.012 to 2.75 ft., and the widths of channel, from 0.9 to 4.2 ft. Included in the paper are the results of 2 438 separate volumetric measurements of discharge for 1 512 different heads, all made at Cornell University and now published for the first time. Of these, 1 276 measurements for 530 heads were made by the writers. The distribution of the velocities in the channel of approach was measured by current meter for 241 heads (196 by the writers), including many cases with abnormally high velocities near the surface and near the bottom. The experiments were made at various times between 1904 and 1920, with the major part between 1913 and 1916. A condensed summary of the experiments is given in Table 1.

Findings.—The main findings are:

(1) Considerable limitation should be understood as to the widespread idea that a precision in discharge estimates within 1% or 2% is obtainable

Note.—Written discussion on this paper will be closed in **January, 1928.**

* Prof. of Experimental Hydraulics, Cornell Univ., Ithaca, N. Y.

† Mr. Turner, formerly Assistant Professor of Hydraulics, School of Civil Engineering, Cornell University, died October 21, 1918. The paper is presented by Professor Schoder. The writers jointly had completed all the experimental work, had made the primary calculations, and had reached the main conclusions in a general way, although not as far as many refinements and important comparisons are concerned. Only the first of the formulas involving v_a (see p. 1434) had been suggested or tested.

TABLE 1—CONDENSED SUMMARY OF WEIR EXPERIMENTS.

All Weirs Full Width of Channel (Without End Contractions). Every Discharge Measurement Volumetric.

Series.	Date.	Height of weir, in feet.	Length of weir, in feet.	Approximate range of heads, in feet.	Number of separate volumetric measurements.	Number of different heads.	Number of runs with velocity distribution measured.	Remarks.
Schoder and Turner A....	Feb. 2-Mar. 11, 1911	7.53	4.22	0.05 to 2.44	63	27	10	Old iron plate.
" " B....	Mar. 25-31, 1911	5.51	4.22	0.05 to 2.44	69	26	11	New iron plate.
" " C....	Feb. 8-Apr. 12, 1914	5.34	4.22	0.036 to 2.53	127	60	7	Part of old crest from 16-ft. canal.
" " D....	Dec. 27, 1914-Jan. 2, 1915	7.50	4.22	0.028 to 2.75	111	40	16	12-in. rolled brass plate carefully maintained.
" " E....	Jan. 4-9, 1915	0.50	4.22	0.029 to 2.00	57	29	11	
" " F....	Jan. 10-15, 1915	1.00	4.22	0.029 to 2.36	91	37	17	
" " G....	Jan. 24-28, 1915	1.50	4.22	0.018 to 2.41	90	37	15	
" " H....	Jan. 30-Feb. 4, 1915	2.00	4.22	0.022 to 2.47	87	34	14	
" " I....	Feb. 5-7, 1915	3.00	4.22	0.022 to 2.67	93	36	15	
" " J....	Feb. 8-11, 1915	4.00	4.22	0.017 to 2.70	93	36	15	
" " K....	Feb. 12-24, 1915	5.50	4.22	0.013 to 2.65	100	38	16	
" " L....	Feb. 28-Mar. 1, 1915	7.50	4.22	0.018 to 2.24	37	24	3	
" " M....	Feb. 26-Mar. 4, 1916	7.50	4.22	0.014 to 2.53	72	31	9	
" " N....	Mar. 26-Apr. 16, 1916	0.75	4.22	0.012 to 2.33	100	42	23	
" " O....	Apr. 17-23, 1916	5.50	4.22	0.05 to 2.71	74	29	10	
" " P....	Jan. 13-18, 1920	2.50	4.22	1.01 and 2.04	12	4	4	
Dawson I.....	Jan , 1913	5.51	4.22	0.10 to 2.54	39	18	...	Schoder and Turner B after 2 years.
" II	May, 1913	5.49	4.22	0.11 to 2.65	53	16	...	New sharp,
" III.....	June, 1913	5.49	4.22	0.11 to 2.08	33	10	...	Crest II after 6 weeks.
Martin.....	Jan.. 1906	1.20	2 023	0.095 to 1.064	30	30	...	New crest.
Meyer and See 1 A.....	Mar. 26-Apr. 2, 1914	2.50	2.02	0.05 to 1.34	106	63	...	Brass crest.
" " 1 B.....	May 12, 1914	2.287	2.02	0.04 to 1.32	38	38	...	" "
" " 1 C.....	May 14, 1914	2.068	2.02	0.05 to 1.33	37	37	3	" "
" " 1 D.....	May 16. 1914	1.742	2.02	0.05 to 1.32	37	37	...	" "
" " 1 E.....	May 16-18, 1914	1.409	2.02	0.05 to 1.30	38	38	3	" "
" " 1 F.....	May 19, 1914	0.975	2.02	0.04 to 1.28	38	38	...	" "
" " 1 G.....	May 19-21, 1914	0.755	2.02	0.05 to 1.25	37	37	3	" "
" " 1 I.....	May 23, 1914	0.000	2.02	0.10 to 0.95	36	36	...	" "
" " 2 B.....	Apr. 17, 1914	0.198	2.02	0.05 to 1.12	36	36	...	" "
" " 2 C.....	Apr. 18, 1914	0.406	2.02	0.044 to 1.17	38	38	...	" "
" " 2 D.....	Apr. 25, 1914	0.723	2.02	0.052 to 1.26	38	38	...	" "
" " 2 E.....	May 2, 1914	1.003	2.02	0.047 to 1.29	37	37	...	" "
" " 2 F.....	May 5, 1914	1.409	2.02	0.046 to 1.31	37	37	...	" "
" " 2 G.....	May 7, 1914	1.807	2.02	0.052 to 1.30	38	38	...	" "

TABLE 1—(*Continued*).

TABLE 1—(*Continued*).

Series.	Date.	Height of weir, in feet.	Length of weir, in feet.	Approximate range of heads, in feet.	Number of separate volumetric measurements.	Number of different heads.	Number of runs with velocity distribution measured.	Remarks.
J. O. Jones A	Apr., 1915	2.50	2.018	0.0196 to 1.359	42	32	3	
" " B	" "	2.50	2.018	0.02 to 1.357	31	28	...	
" C	" "	2.50	2.018	0.019 to 1.357	32	28	3	
" D	" "	2.50	2.018	0.016 to 1.355	36	29	3	
" E	" "	2.50	2.018	0.020 to 1.354	29	27	3	
" F	" "	2.50	2.018	0.021 to 1.353	29	27	3	
" G	" "	2.50	2.018	0.020 to 1.353	29	27	3	
" H	" "	2.50	2.018	0.017 to 1.356	38	29	3	
C. M. Weber II	May and June, 1915.	2.50	2.018	0.021 to 1.356	37	32	3	
" III	" " "	2.50	2.018	0.025 to 1.358	34	29	3	
" IV	" " "	2.50	2.018	0.018 to 1.355	30	27	3	
" V	" " "	2.50	2.018	0.017 to 1.353	29	26	3	
" VI	" " "	2.50	2.018	0.089 to 1.363	20	19	3	
Total number volumetric measurements made at Cornell University now published for the first time					2 438	1 512	241	
THE FOLLOWING DATA COVER THE CLASSIC EXPERIMENTS USUALLY QUOTED IN AMERICA.								
Fteley and Stearns*	Mar., 1877	8.17	5.00	0.07 to 0.82	24	24	...	Omitting doubtful runs.
" "	Nov.–Dec., 1879	6.55	19.00	0.47 to 1.603	9	9	1	
Bazin† Series I	Sept.–Oct., 1886	3.72	6.56	0.19 to 1.012	67	67	...	
" " II	Oct., 1886	3.72	3.28	0.187 to 1.338	38	38	...	
" " III	Nov.–Dec., 1886	3.297	1.64	0.19 to 1.78	48	48	...	
J. B. Francis	Oct.–Nov., 1852	5.048	9.997	0.62 to 1.55	46	5	...	Two end contractions.
" "	Nov., 1852	5.048	7.997	1.018	2	1	...	Four end contractions.
" "	Oct.–Nov., 1852	2.014	9.997	0.65 to 1.05	19	3	...	Two end contractions.
" "	Nov., 1852	5.048	9.995	0.795 and 0.979	12	2	...	Expansions down stream.
" "	Nov., 1852	5.048	9.995	1.00	5	1	...	Number of expansions or contractions.
" "	Oct.–Nov., 1852	2.014	7.997	0.680	4	1	...	Four end contractions.

* Fteley and Stearns in addition to their volumetric measurements ran 94 experiments using comparative discharges (not volumetric) with 21 heads on weirs of various heights above the bottom of the channel of approach, which were 0.5, 1.00, 1.70, 2.60, and 3.56 ft. These were without end contractions. In addition, 5 heads, 18 experiments with 5 heads, were run on weirs with end contractions.

† In addition, Bazin Series IV to X, 228 experiments were made with comparative discharges as calculated from his standard weir which was calibrated by his Series I, II, and III.

merely (*a*) by careful observations of the head of water on a weir having nominally a sharp square edge, but built without precise regard to the degree of sharpness and smoothness of the crest plate and to the velocity conditions in the channel of approach; and (*b*) by the use of the standard formulas or other estimating devices of present engineering practice.

(2) The use of the simple Francis weir formula:

$$Q = 3.33\ L\ h^{\frac{3}{2}}, \text{ or } Q = 0.623 \times \frac{2}{3}\ L\ h\ \sqrt{2\,g\,h}$$

as a basic formula is substantiated for cases closely approximating the ideal weir with a truly sharp, square edge, with smooth, vertical, up-stream face near the crest, and with a deep up-stream pool, or with negligible effect of velocity of approach. (For low heads, see Item (5)).

(3) This paper shows the extent and nature of the inadequacy, for general precise work, of formulas that introduce merely the mean velocity of approach; or that, as the equivalent, introduce merely the height of weir; also the inadequacy of curves and tables that merely average or approximate the data of the experiments hitherto published. This inadequacy consists chiefly in the failure of the experimenters to measure and place on record the distributions of velocities in the channel of approach. The mere statement that velocities were approximately equalized by baffles leaves the matter too uncertain except for a limited range of heads and heights.

(4) The new experimental data (by several experimenters) agree with those of Francis, Fteley and Stearns, and Rehbock, in their evidence that Bazin's coefficients and formula for sharp-crested weirs give too high discharges for low heads—by fully 2 to 3% for heads of 0.3 ft. to 1.2 ft.

(5) For low heads of less than about 0.4 ft. for weirs more than 2 ft. high: (*a*) an additive corrective expression of the Fteley and Stearns' type, but made to the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}} + 0.007\ L$, is verified, although their value of 0.007 ft-sec. units is to be replaced by other values ranging from 0.004 to 0.012 in certain cases; (*b*) many data of percentage corrections for extremely low heads (less than 0.10 ft.) on a variety of crest pieces are graphically shown.

(6) The percentage increase in discharge (as compared with sharp-crested conditions) due to a slight rounding of the up-stream top corner of the crest (say, to as much as $\frac{1}{24}$-in. radius = about 1 mm., which may soon occur with exposed steel and which has been overlooked or neglected in cases intended to be "sharp-crested") is shown to be as much as 2% for heads of 0.50 ft. and 0.5% for heads of 1.35 ft. A rounding of $\frac{1}{8}$-in. radius causes about 3%, and one of $\frac{1}{4}$-in. radius, about 5½%, increased discharge at 0.50-ft. head. Agreement is found with Fteley and Stearns' data for the larger roundings.

(7) The percentage increase in discharge due to changing the roughness of the up-stream face of the weir bulkhead from that of a polished brass plate to that of a coarse file for a distance of 12 in. below the crest is shown to range from about 2% for 0.50-ft. head to about 1% for 1.35-ft. head. This agrees with the previous findings of James Barr for V-notched weirs.

(8) The new data show no justification for a taboo on a weir with a head of water as great as, or greater than, the height of the crest above the bottom of the channel. When the use of such a weir is convenient, the new formula, involving auxiliary velocity measurements, and several simpler formulas are amply accurate. In fact, there seems to be an inherent certainty of action for the very high heads on very low weirs.

(9) While the importance of careful measurements of heads, including the precise referencing of the gauge to crest level (zero determination) and the use of appropriate instruments, are not to be minimized, yet all the writers' work substantiates the conclusions of earlier experimenters that there is no necessity for slavish imitations in the precise manner of determining the head, nor for a precise location of station up stream from the weir to agree with a particular experimenter, provided no violence is done to the general hydraulic circumstances. Hitherto, many errors have been charged to insignificant details of the head determination when the trouble has been due to the lack of appreciation of other factors and to neglect in making proper arrangements or measurements.

As a general rule, rarely subject to exceptions, the head should be determined in one of two ways: (1) by a direct measurement in the channel of approach (by one of several appropriate devices); or (2) by a piezometer with a stilling-well such that no correction is necessary.

(10) For precision it is necessary to deal not (as in most formulas) with the head corresponding to the mean velocity of approach, but with the head corresponding to the velocity above the level of the weir crest. This involves auxiliary measurements by current meter, Pitot tube, or floats, etc.

Where it is possible actually to duplicate the definitely known velocity conditions prevailing for different heads and heights in the channel of approach, the use of a special formula or special coefficients fitting the particular case remains entirely proper.

The extreme difference in discharge due to altering the distribution of velocities up stream from a weir, keeping the head constant, amounted to 26 per cent. The auxiliary velocity measurements by the writers and the introduction of this new factor are, in fact, simply carrying to a logical conclusion the suggestions of J. B. Francis, Hamilton Smith, Fteley and Stearns, Bazin, and others.

(11) A tentative formula,

$$Q = 3.33\, L \left[\left(h + \frac{V a^2}{2 g} \right)^{\frac{3}{2}} + \left(\frac{V b^2}{2 g} \times h \right) \right]$$

is suggested, in which, $\frac{V a^2}{2 g}$ is the mean velocity head of approach above the crest level, and $\frac{V b^2}{2 g}$ the mean velocity head of approach below the crest level. (See Item (14).) If the velocity conditions are unknown it is necessary to make measurements to find the distribution. This formula is tested by its application to the new experimental data and is found to fit all of them much better than the formulas of Fteley and Stearns, or of Bazin, or the curves of

Lyman or the formula, $Q = 3.33\ L \left(h + \frac{V a^2}{2 g}\right)^{\frac{3}{2}}$. In extreme cases of high heads on low weirs the corrective terms modify as much as 50% the value that would be obtained by the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}}$, but, nevertheless, they fit the volumetric measurements within about 1 per cent. Other simpler formulas of limited applicability are suggested also. In particular, attention is called to Rehbock's formula, $Q = \frac{2}{3}\ \omega\ L\ h \sqrt{2\ g\ h}$, in which, $\omega = 0.605 + \frac{1}{320\ h - 3} + 0.08\ \frac{h}{d_0}$, in foot-second units ($d_0$ is the height of the weir).

(The experimental data of Francis, Fteley and Stearns, Bazin, etc., cannot be used to test the new formula because no adequate velocity measurements were made).

(12) The use of such a weir properly built, of good head measurements and good auxiliary velocity measurements in the channel of approach, should insure discharge values within 2% of the truth, and in most cases within 1% (see Fig. 18). However, this seems to be quite untrue for many weirs relied on for such a precision by the use of formulas and methods in common use.

(13) The paper in no way presumes to champion the sharp-crested weir as a measuring device superior to others.

Experimental Equipment

The Experimental Canal.—The weirs for the experimental series, A to P, inclusive, (Fig. 1) were in the Hydraulic Laboratory of Cornell University. Inside a concrete canal, 6 ft. wide, was formed a new inner wooden canal approximately 4 ft. 3 in. wide, with flared sides up stream forming an easy approach for the water; and with down-stream sides projected beyond the weir to prevent lateral spreading of the overfalling water. Aeration holes just below, and close to, the crest on the down-stream side served to admit air under the overflowing sheet of water for the low heads. For the higher heads the general construction permitted free access of air under the sheet. Other essential details of the canal are given in Fig. 1.

Water Supply.—The water supply came by gravity from Beebe Lake, a pond of 20 acres formed in Fall Creek by a concrete overfall dam, 26 ft. high, with a spillway 130 ft. wide. During the experiments the water was usually flowing over the spillway, thus insuring nearly a constant head. The watershed was 125 sq. miles. From the base of the dam the water flowed through 192 ft. of 48-in. pipe and then 52 ft. of 30-in. pipe, terminating at the side of the small canal near the up-stream end with its invert even with the bottom of the canal (Fig. 1). For regulating the flow there was a 30-in. hand-operated gate-valve. The static head on the center of the valve with water at the crest of the dam was 17.5 ft. With water in the small canal barely flowing over the experimental weir, the back pressure in the canal reduced the difference between the head up stream and down stream from the 30-in. valve to an average of about 10 ft.

Baffles.—Instead of allowing the discharge from the valve and the 30-in. pipe to churn around unrestrictedly in the canal, and depending on the usual type of baffle-racks to quiet the flow, a deflecting hood was constructed, to serve both as an enlarging elbow and as an energy killing chamber for the first and most violent part of the commotion (Fig. 1). The water flowing out under the hood was further quieted and directed, first upward, then forward over the top of the hood by a system of open deflector baffles arranged so as to enlarge the cross-section progressively. This scheme proved remarkably effective except for some tendency to form surface waves at the very highest flows (heads above 2.0 ft. on the weir). A stilling-raft of 2-in. planks floating on top of the water above the deflectors tended to eliminate these waves.

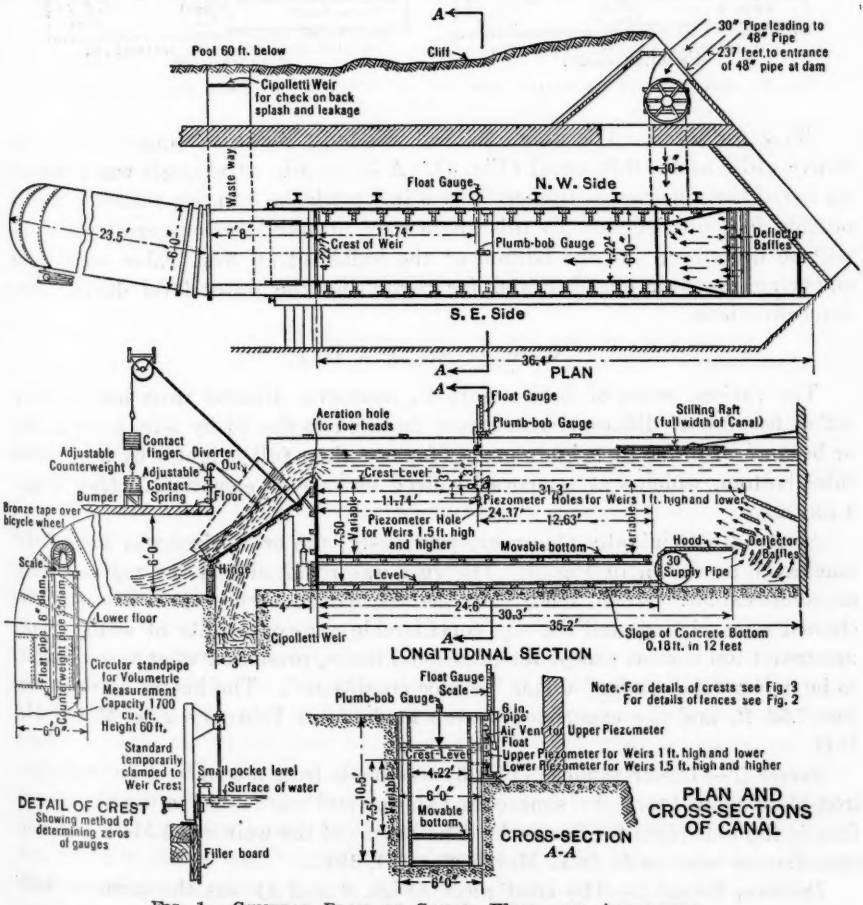

FIG. 1.—GENERAL PLAN OF CANAL, WEIR, AND APPARATUS.

"Fences."—In order to vary the distribution of velocities from the conditions during the so-called "regular" runs with only the permanent baffles controlling the flow, additional controls were built across the canal for some of the higher heads. These were usually in the form of inclined fences (Fig. 2)

with 1 by 6-in. horizontal boards spaced by trial at such distances as to secure the desired deflection of a part of the water toward the surface or toward the bottom, as the case might be, always allowing some of it to flow through the open spaces.

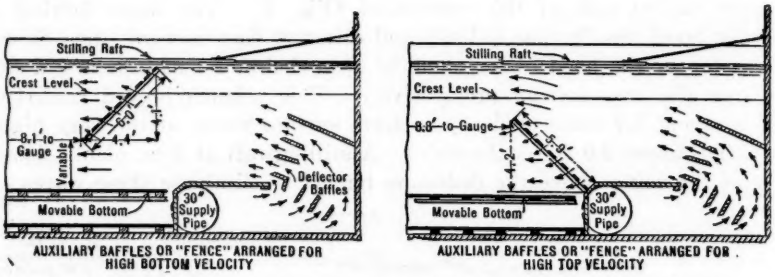

FIG. 2.—LOCATION OF AUXILIARY BAFFLES OR "FENCES" TO VARY THE DISTRIBUTION OF VELOCITY.

Weir Bulkhead.—The weir bulkhead was built vertically square across the entire width of the 6-ft. canal (Fig. 1). A 3½ by 5-in. steel angle was fastened on top of the upper cross timber (4 by 6 in.), with the 5-in. leg vertical and its outside face up stream. To this angle were attached various crest pieces as will be described. At the bottom of the bulkhead an 8-in. valve served for emptying the weir chamber and for regulating the water level during zero determinations.

Weirs of Series A to P

The various weirs of Series A to P, inclusive, differed from one another either because of different crest pieces fastened to the 3½ by 5-in. steel angle, or because of different heights made (for less than full heights) by a movable false bottom, which was temporarily fixed at various elevations. (See Figs. 1 and 2.)

Series A.—Originally (ten years previously) the crest piece was accurately machined, as shown in Fig. 3. The rust was removed and the top was filed to some extent. When ready for the experiments the up-stream surface showed some pitting, and the top, considerable non-conformity of width. The up-stream top corner, except for occasional dents, presented what was thought to be a fair "sharp edge" under "service conditions". The height of the weir was 7.53 ft. and the experiments were made from February 2 to March 11, 1911.

Series B.—The crest piece (Fig. 3) was made from a ⅜-in by 4-in. wrought-iron plate, planed straight, square, and sharp; and was bolted to the up-stream face of the weir plate for Series A. The height of the weir was 5.51 ft. and the experiments were made from March 25 to 31, 1911.

Dawson, Series I.—The crest piece (Figs. 3 and 4) was the same as that used in Series B, but after two years' service. The up-stream face and top appeared to have suffered considerable corrosion, although the up-stream top corner showed unevenness rather than rounding. It was more decidedly pitted than the crest piece of Series C. The height of the weir was 5.51 ft. and the experiments were made in January, 1913.

Dawson, Series II.—A $\frac{3}{8}$-in. by 4-in. wrought-iron plate was accurately planed on top and along the up-stream face. The up-stream top corner was square and sharp; the top was flat, $\frac{1}{8}$ in. wide, with its down-stream edge rounded to a $\frac{3}{8}$-in. radius. (See Fig. 3.) The experiments were made while the crest was sharp, smooth, and free from rust. The height of the weir was 5.49 ft. and the experiments were made in May, 1913.

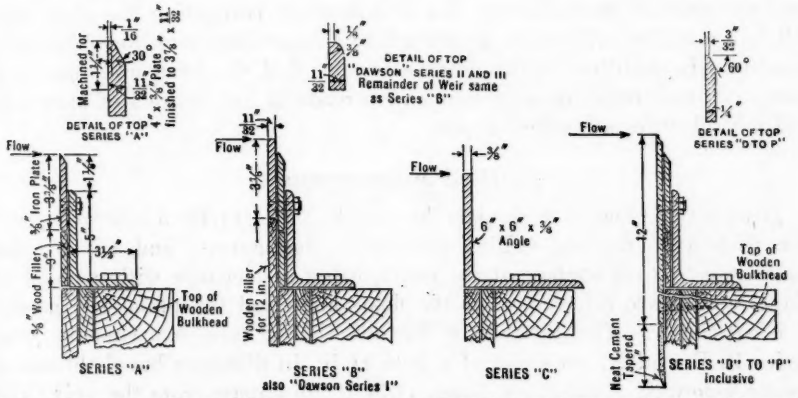
DETAIL OF TOP SERIES "A"
DETAIL OF TOP "DAWSON" SERIES II AND III Remainder of Weir same as Series "B"
DETAIL OF TOP SERIES "D TO P"
Flow
Top of Wooden Bulkhead
Wooden filler for 12 in.
6" x 6" x ⅜" Angle
Neat Cement Tapered
SERIES "A"
SERIES "B" also "Dawson Series I"
SERIES "C"
SERIES "D" TO "P" inclusive

FIG. 3.—DETAILS OF CRESTS, SERIES A, B, C, AND D TO P, ALSO DAWSON SERIES, I, II, AND III.

Dawson, Series III.—The plate and setting was that used in Series II (Fig. 3), but after six weeks' exposure to water. About ten rust nodules could be detected on the up-stream top corner of the crest and slight roughness could be felt. The height of the weir was 5.49 ft. and the experiments were made in June, 1913.

Series C.—A 6 by 6-in. by $\frac{3}{8}$-in. angle (Figs. 3 and 5), a part of the crest piece from the lower 16-ft. weir in the outside experimental canal served as a crest. This angle was used in place of the 5 by $3\frac{1}{2}$-in. angle previously described. The up-stream face and top were originally planed straight and sharp with a flat top $\frac{3}{8}$ in. wide. The experiments were made after the crest had been in service about twelve years, during which time it was frequently scrubbed and occasionally filed to keep the up-stream face and the up-stream top corner free from rust. During the Series C measurements, the crest part of the angle (see Fig. 5) was in a good state of preservation, showing extensive rusting on the up-stream face near the top, but not what would be called a pitted surface. The top had somewhat of a glaze, making it smooth to the touch. The up-stream top corner of the crest had lost its extreme sharpness so that, while for the most part square and sharp to the casual inspection, it was actually dented and felt dulled and rounded. The height of the weir was 5.34 ft. and the experiments were made from February 8 to April 12, 1914.

Series D to P.—The crest piece was of rolled brass, 12 in. wide and $\frac{1}{4}$ in. thick, and was added to the up-stream face of the crest piece, Series A. The up-stream top corner was straight, square, and sharp. The flat top was carefully machined to a $\frac{3}{32}$-in. width and beveled down stream 60° to the horizontal. During the experiments a true sharp corner was carefully maintained

by occasional use of a fine carborundum stone. The up-stream face was kept smooth and free from organic deposits. The small shoulder at the lower edge of the plate was filled in with neat cement for a 4-in. depth and tapered to a thin edge. (See Fig. 3.)

METHOD OF EXPERIMENTATION

General Method.—The procedure was to establish a steady flow of water over the weir at approximately the desired head, permitting the discharge to fall into a wasteway, then to divert it into a measuring tank for a determined interval. In addition to the observations to find the head and rate of discharge, current-meter measurements were made in the channel of approach to find the velocities at various depths.

Head Measurements

Head measurements were made by two devices: (1) By a side-wall piezometer with a connecting stilling-well for a float-gauge; and (2) by direct observations on the surface of the water, using a steel tape with a plumb-bob. These devices are referred to as the float-gauge and the plumb-bob gauge.

Float-Gauge.—The piezometer for all the writers' experiments, except Series E, F, and N, consisted of a hole 1⅛ in. in diameter bored through the wooden side-wall of the inner canal, 11.74 ft. up stream from the weir (which distance is about four times the highest head) and 1.25 ft. below the crest level (Fig. 1). For Series E, F, and N the piezometer consisted of two such holes (Fig. 1), 9 in. apart horizontally, located 0.25 ft. below the crest level, the mid-point between the holes being vertically above the single hole just described. This different piezometric arrangement was made necessary by the fact that the floor of the channel for Series E, F, and N was above the single piezometer hole. It was deemed desirable to use the same float-gauge for all series, and there was an advantage in construction in having two symmetrically placed holes, with a supposed greater accuracy of readings for these shallow channels with relatively high velocities of approach.

Into the single hole was screwed, horizontally from the outside, a 20-in. length of 1-in. standard wrought-iron pipe, but not projecting through on the inside. At its outer end an up-turned elbow connected it with a vertical, 6-in. wrought-iron pipe, 5 ft. high, serving as a stilling-well for the float-gauge proper (Section *A-A*, Fig. 1).

In the well was a cylindrical copper float, top and bottom ends dished outward so as to shed the water and not to entrap air. The float stem consisted of a ¼-in., round, brass rod passing axially entirely through the float, thus materially stiffening it.

The stem was long enough to extend above the top of the 6-in. pipe when the water level in the canal was several inches below the weir-crest level. At the top of this stem a combined index and guide (Fig. 6), traveled along the edge of a brass scale graduated to hundredths of a foot.

For Series A, B, and for 18 runs of Series C, the float was a cylinder, 5⅝ in. in diameter. For all later work, a 5-in. float was used with four vertical fins attached. These were rounded off so as to project ⅜ in. at the middle

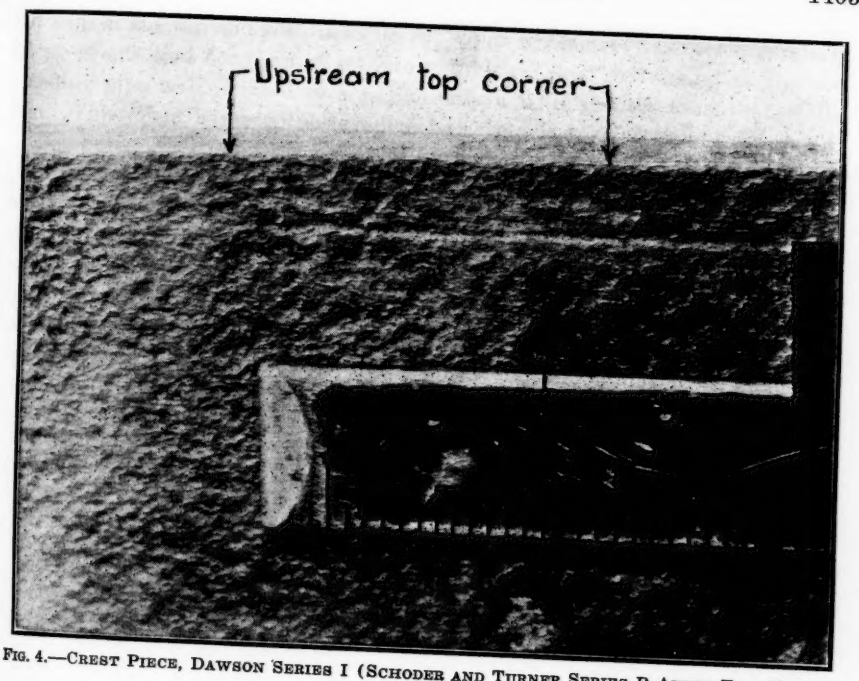

FIG. 4.—CREST PIECE, DAWSON SERIES I (SCHODER AND TURNER SERIES B AFTER TWO YEARS).

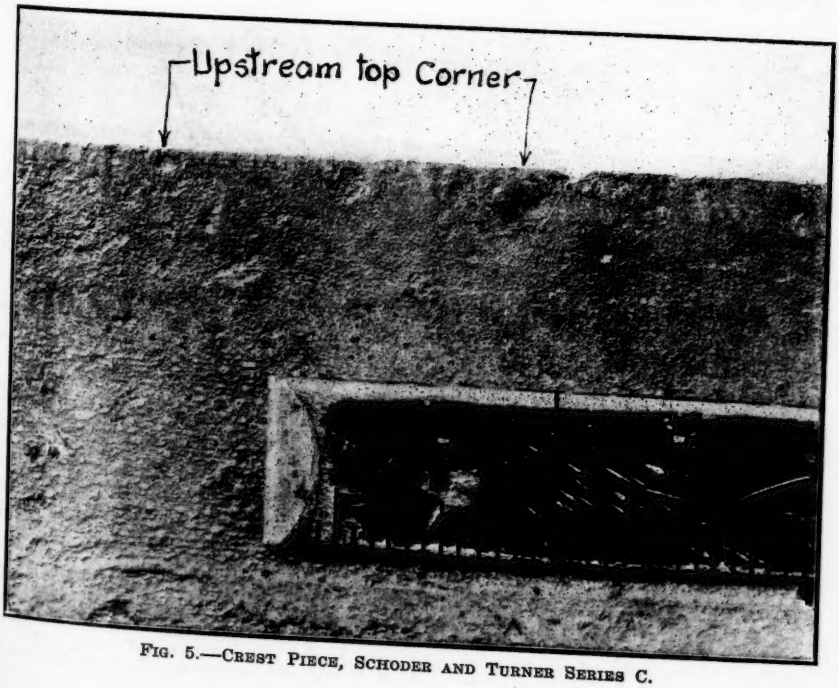

FIG. 5.—CREST PIECE, SCHODER AND TURNER SERIES C.

and ¼ in. at the top and bottom (Fig. 6), which arrangement avoided capillary lifting effects that occur when a plain cylindrical float drifts against the surrounding pipe walls curved to a radius only a little greater than that of the float. This effect has been observed to be as much as 0.003 ft.

For Series E, F, and N, a pair of horizontal 1-in. pipes led from the two piezometer holes to tees in vertical 1-in. pipes, the downward extensions of which on either side led through an elbow and a short horizontal piece to a cross in the 1-in. pipe leading out of the lower piezometer hole. The upward extensions of the vertical pipes reached above the highest water level in the canal and served as air vents. For the lower piezometer the float well itself served as an air vent. Plug cocks in the three 1-in. pipe lines enabled either the lower single-hole piezometer or the upper double-hole one to be connected to the float well.

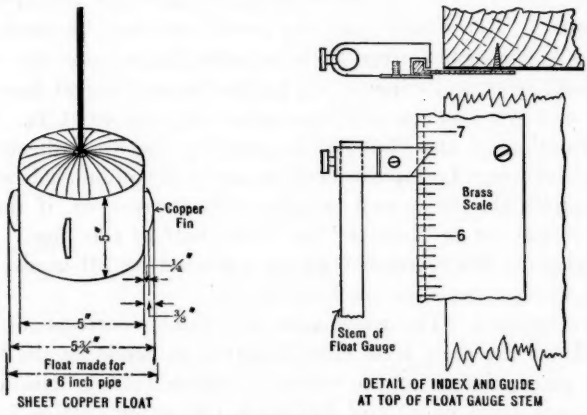

FIG. 6.—FLOAT-GAUGE DETAILS.

In use, the float-gauge has the great merit of giving continuous indications without the necessity of making settings. The degree of damping of the magnitude of waves or surges may be varied by changing the ratio of area of piezometer hole to that of the float well. To remove any mechanical friction or "sticking" of the index the observer at short intervals rapped the framework of the scale.

With high heads, or rapid flows in the channel of approach, the alternating high and low readings of the float-gauge were taken. Usually the time interval between successive high and low indications was 15 to 40 sec. As a general rule, several readings of the float-gauge were taken just before, during, and after the discharge measurement.

*Plumb-Bob Gauge.**—This gauge (Fig. 1) consisted of a steel tape, graduated to hundredths of a foot, to the end of which was attached a brass plumb-bob. A 4 by 12-in. plank, securely bolted, across the 6-ft. canal carried the "tape block", having on its top a brass plate grooved to admit the steel tape

* The plumb-bob gauge used was substantially as designed by Gardner S. Williams, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1261.

and marked with an index line. The tape hung over the rounded end of the block the same distance up stream from the weir as the piezometer holes of the float-gauge. In use, the plumb-bob was rapidly lowered by unreeling the tape which was then slipped into the groove where it was lightly held with the finger tips or thumb of the left hand. The right hand steadied by the back of the tape block then fed the tape down or up, as desired by manipulation between thumb and forefinger. The bob was slowly raised out of the water until the "bubble" at the point "broke" (that is, disappeared), the point then being about 0.005 ft. above the water surface. The tape was then very slowly fed out until the "bubble" formed at the point of the bob indicating contact with the surface, for which position the reading was taken. This phenomenon was of pronounced visibility, even when the observer was 10 or 15 ft. above the water surface.

The device proved to be far more sensitive than one unacquainted with it would imagine. Repeatedly students would convince themselves to their surprise that it was possible, after 3 min. of practice, to make the tape "jump" forward or backward (as distinctly felt by the finger tips) at least three successive times and yet effect a total movement of only 0.001 ft. With quiet water the formation of the "bubble" appeared to be an indicator of corresponding sensitiveness. In rapidly flowing water the plumb-bob was adjusted to show alternately the crests and troughs of the waves, or, if these were of small height, it was set to "scratch" the water half of the time. For several series of experiments the plumb-bob gauge was read for all heads, but, in the later series, it was not read for the high heads.

Zero Determinations.—The readings of the gauges corresponding to water surfaces exactly at the crest level (zero head) were made by the "water-level method". By manipulation of the supply or waste valves, depending on the amount of leakage from canal and bulkhead, the water surface in the canal was held practically steady just below the level of the crest or perhaps a few thousandths of a foot above the crest.

A hook-gauge, temporarily clamped to the crest (Fig. 1), served to measure the exact difference of elevation between water surface and the crest. The point of the hook was adjusted so as to be about 1½ in. up stream from the crest. A small pocket level with a sensitive bubble was nearly balanced across the crest piece with the up-stream end resting lightly on the point of the hook. The hook-gauge was then adjusted so as to center the bubble, and the hook-gauge reading was taken. The procedure was repeated with the spirit-level reversed. Then, on signal from the hook-gauge observer, when he made contact with the water surface, the float-gauge was read. This was repeated six times and then the crest-gauge was again read for crest level.

The whole procedure was repeated for the plumb-bob gauge (there being but two observers). Such a determination was made with the hook-gauge at the middle of the crest and also at the one-sixth points out from either canal wall, so that there was an independent zero determination for each one-third of the crest length for each gauge. For some series several such complete determinations were made.

The benefits of these careful zero determinations appear in the close agreement of heads as determined by the two types of gauges. (See summary in Table 2.)

Discharge Measurements

The discharge measurements of the sheet of water flowing over the weir were obtained by diverting it for an appropriate interval of time from the wasteway into the calibrated tank.

"Diverter."—The diverter (Fig. 1) was an inclined rectangular wooden apron, 8. ft. long, fitted with sheet-iron sides, 15 in. high, so as to form a steep chute. It was hinged near its lower edge to the top of a bulkhead built at the entrance to the standpipe.

Hoisting Tackle.—This consisted of a 1-in. rope passing over a pulley, the up-stream end being attached to the apron by rod-links and the other end sustaining an adjustable counterweight.

TABLE 2.—CORRESPONDENCE BETWEEN "HEADS" MEASURED (1) BY FLOAT-GAUGE; AND (2) BY PLUMB-BOB GAUGE FOR VERY LOW HEADS, SHOWING THE CLOSE AGREEMENT BETWEEN THE TWO METHODS.

(Heads below 0.100 ft. for Series A, B, and D to P; heads below 0.200 ft. for Series C.)

Series D to P, Inclusive:		
Number of runs where plumb-bob gauge was higher than float-gauge =	70	
Average head, in feet, that these runs were " " " " =		0.0004
Number of runs where plumb-bob gauge was lower " " " =	39	
Average head, in feet, that these runs were " " " " =		0.0004
Number of runs with the same head by both gauges..... =	5	
Total number of runs considered in this comparison..... =	114	
Number of runs with 0.0005 ft., or less variation = 92		
" " " " 0.0010 ft., " " " = 108		
Maximum variation = 0.0016 ft.		
Series A, B, C:		
1st.—Using old type float (without fins):		
Number of runs where plumb-bob gauge was higher than float-gauge =	9	
Average head, in feet, that these runs were " " " " =		0.0011
Number of runs where plumb-bob gauge was lower " " " =	16	
Average head, in feet, that these runs were " " " " =		0.0008
Number of runs with the same head by both gauges..... =	1	
Total number of runs considered in this comparison..... =	26	
Number of runs with 0.0005 ft., or less variation = 11		
" " " " 0.0010 ft., " " " = 19		
Maximum variation = 0.0022 ft.		
Series C:		
2d.—Using new type float (with fins):		
Number of runs where plumb-bob gauge was higher than float gauge =	14	
Average head, in feet, that these runs were " " " " =		0.0005
Number of runs where plumb-bob gauge was lower " " " =	15	
Average head, in feet, that these runs were " " " " =		0.0002
Number of runs with the same head by both gauges..... =	2	
Total number of runs in this comparison..... =	31	
Number of runs with 0.0005 ft., or less variation = 24		
" " " " 0.0010 ft., " " " = 29		
Maximum variation 0.0011 ft.		

A 1-ton spur-geared chain block served to hoist the counterweight and to sustain it until after the diverter was dropped from the "up" to the "down" position. It also served as a control while adjusting the diverter contact.

To hold the diverter in the "up" position when the counterweight was hoisted, the bent end of an iron rod was "hooked" over the lip of the diverter,

the other end being hooked over an iron railing. This holding rod was called the "trigger". A small rope, tied to the trigger near the lower end, passed over the top of the extension side of the inner canal where it sustained a small counterweight. A pull on the diverter rope by one of the experimenters, bracing himself against the railing, raised the diverter slightly so that the trigger could be pulled out of the way by the small counterweight. The diverter could then be "dropped" through the sheet of water, and the large counterweight released by the chain block until it hung free. This counterweight had been adjusted previously so that the force of the sheet of water would hold the diverter down, but a slight downward pull on the counterweight would cause the diverter to rise through the sheet of water.

The diverter chute was 5.5 ft. wide, being narrow enough to have some clearance inside the 6-ft. canal and inside the entrance to the 6-ft. standpipe, and yet wide enough to project about 6 in. outside the over-head extensions of the side-walls of the inner canal. When the diverter was in the "down" position, it was supported by a "bumper" consisting of a 4 by 4-in. cross-piece resting on side posts independent of the weir bulkhead. The upper edge or lip of the diverter was about 1 in. below the down-stream edge of the small fixed sloping apron on the weir bulkhead. This arrangement caught all the water, even at low heads, except those between about 0.60 and 0.75 ft., which had to be avoided because the sheet of water struck the diverter so close to the upper edge that considerable water splashed backward and upward through the clearance space between the diverter and the bulkhead apron.

Wasteway.—The wasteway (Fig. 1) acted as an intercepter for any leakage from the weir bulkhead, so that the measuring tank was unaffected by such leakage. With the highest heads, in Series A, B, C, and Dawson's Series I, II and III (more than 2 ft. over the weir), when the diverter was in the "up" position with water flowing to waste, there was considerable turmoil in the wasteway, and a small quantity of water found its way into the stand-pipe through the hinge and over the bulkhead through the clearance spaces outside the diverter. This small leakage was accounted for in the regular course of the observations for initial and final water levels in the stand-pipe measuring tank. When the diverter was in the "down" position (water flowing into the stand-pipe), there was, at the highest heads of Series A, B, and C, a small back-splash through the clearance spaces out from the standpipe into the wasteway. To account for this, the law of total leakage into the wasteway was found by observing the heads on the 12-in., Cipolletti weir notch (Fig. 1) in a bulkhead across the outer end of the wasteway channel, for various heights of water back of the experimental weir, ranging from several feet below the weir crest to the level of the highest heads used in the experiments.

For water levels above the crest of the weir the diverter was in the "down" position, as during the discharge measurements. Thus, the law of leakage variation through the bulkhead for different heads was clearly established, and any excess was charged to back-splash from the stand-pipe. As a check, actual observations of conditions were made by a man going into the wasteway under the diverter while the flow was proceeding. Both the weir bulkhead and the

bulkhead at the stand-pipe entry were frequently examined, and conditions always found to be as assumed for the correction measurements and calculations.

This back-splash from the stand-pipe into the wasteway at high heads was practically eliminated for the writers' Series D to O, by sealing all clearance cracks of appreciable size with thin wooden strips.

Stand-Pipe Measuring Tank.—This tank (Fig. 1) was a circular steel stand-pipe, 6 ft. in diameter, 60 ft. high, made of cylindrical sections, 5 ft. high. It was calibrated in two ways: (1) By measuring its outside girth by a steel tape with proper allowance for thickness of metal, vertical position of the lap joints, volumes of recesses inside at various attached saddle nozzles with their gate-valves, rivet heads, etc.; and (2) by repeatedly filling with water and then draining off successive quantities into a precisely calibrated tank with a capacity of 400 cu. ft. The initial level and the intermediate levels were varied so that the final calibration points covered the entire 60-ft. vertical range by 1-ft. intervals. These two calibrations were referred to tape and scale readings on the stand-pipe float-gauge.

Stand-Pipe Float-Gauge.—The float was of copper, torpedo-shaped, moving in a 6-in. vertical pipe set alongside the stand-pipe and connected to it at the bottom by a 2-in. pipe. A bronze ribbon tape was attached to the top of the float and passed up out of the 6-in. pipe, over a bicycle wheel and down into a 3-in. vertical pipe which served as a guide for the counterweight on the other end of the tape (Fig. 1). On top of the 6-in. pipe, alongside the vertical tape, was mounted an auxiliary brass scale divided into 0.1 and 0.01 ft. The bronze tape had graduations at 1-ft. intervals. This arrangement permitted readings by estimation to 0.001 ft., which extent of change of level in the stand-pipe and float pipe was readily indicated by a corresponding movement of the tape. From time to time the indications of the bronze tape were checked, in operating position, by comparing the indicated difference of elevation between near-full and near-empty conditions with a difference by steel-tape measurements between water gauges near the top and bottom.

The adopted calibration was that by the water-filling method with direct readings on the float-gauge, just as in the experiments proper. This calibration was based on precise weighings of a succession of 5-cu. ft. quantities into the lower 400-cu. ft. rectangular tank for its calibration. This lower tank was also calibrated by dimensional measurements. The two methods agreed within 0.1 per cent.

The precise determination of the inside dimensions of the stand-pipe involved great difficulties, and yet the two calibrations agreed within 0.2 per cent.

Time Measurements.—The durations of the diversions into the stand-pipe were measured, except for low heads (for which an ordinary watch was used) by a ribbon electric chronograph, one pen actuated by a seconds pendulum and the other by a diverter contact.

This contact device (Fig. 1) consisted of a weight, free to move vertically on a pair of fixed iron guide-rods, and suspended by a wire passing over a

small grooved pulley and fastened to the diverter. The weights carried a brass contact finger moving between two leaves of spring brass mounted on a metal block that could be clamped anywhere along a second pair of vertical guide-rods. The position of the gap was adjusted (for each different head on the weir) so that the contact point closed the circuit just as the lip of the diverter passed through the middle of the sheet of water. In some of the first experiments two contacts were used, one adjusted for the diverter lip at the lower, and the other, at the upper, surface of the sheet of water.

With an effective volume of about 1 200 cu. ft. available in the stand-pipe, a measurement at the highest head (60 cu. ft. per sec.) covered 20 sec. With a head of 1 ft., the duration was about 80 sec. Usually, for the highest heads, the discharge was measured five times in succession. The chronograph was used for measurements up to 2 min. in duration. Low-head measurements lasting 3 min. or more were timed by a well-regulated watch, the second hand of which made 5 visible jumps per sec. To avoid effects of eccentricity of the seconds' dial, the runs were made to cover an exact number of full minutes.

Procedure in a Typical Run

As a rule, neither runs nor zero determinations were made until after the weir bulkhead and canal walls had become thoroughly water-soaked. Customarily, water was flowing over the weir at all times, whether or not experiments were in progress.

At the start of a run the 30-in. valve was opened so as to give the desired head as shown by the weir float-gauge. Time was allowed for the head to become steady; this developed almost as fast as the valve could be opened for the high heads, but required as long as 10 or 15 min. for the extremely low heads. The leakage through the weir bulkhead into the wasteway materially hastened the adjustment for these low heads.

The diverter contact was then adjusted to a point midway between temporary chalk marks made on the guide-rods as the diverter was slowly moved through the sheet of water by the chain-block control. Also, the counterweight was regulated to suit the volume of water flowing.

The stand-pipe, having previously been partly filled with water, was now nearly emptied by throwing into the "open" position the lever of a 4-way valve which controlled the 36-in. hydraulic valve at the bottom of the stand-pipe (60 ft. below the operating floor). While the stand-pipe was emptying, the control lever was thrown into the "close" position, the operation being so timed that the 36-in. valve would be completely closed with at least 5 ft. of water remaining in the stand-pipe. This process of closing "under pressure" materially reduced the leakage from the closed valve.

Usually three or more readings at ½-min. intervals were taken on the stand-pipe gauge for the initial water level; also the weir float and plumb-bob gauges were read several times. The chronograph was then started, the diverter dropped, and the counterweight released to hang free. (Usually one observer read the weir float-gauge while the flow was going into the stand-pipe. For low heads both the float-gauge and the plumb-bob tape were read several times

alternately. For the high heads the duration of the volumetric measurement was too short to permit readings of the plumb-bob gauge.) At the end of the desired interval of time the counterweight was pulled, moving the diverter up through the sheet of water, and the trigger was hooked over the lip of the diverter. The chronograph was then stopped and the ribbon examined to make sure the record had no flaws.

Just 1 min. after the diverter went up, allowing this time for settlement in the stand-pipe and its float-pipe, readings were taken for final water level, usually three or more at ½-min. intervals. Immediately after the diverter went up, readings of the weir head-gauges were taken, as at the beginning. The stand-pipe was then emptied to about the 5-ft. level, ready for another run. Some typical original laboratory notes and primary calculations are given in Table 3.

Primary Calculations from Observed Data

Leakage Corrections.—It was the practice to make corrections systematically even when they seem negligibly small, as in most cases. From Column (3) of Table 3, it is seen that in Run 55, Series D, the surface of the water in the stand-pipe measuring tank was rising at the beginning (before the diversion into the tank) at the rate of 0.032 ft. per min., and at the end (after the run) at the rate of 0.025 ft. per min., both values being recorded in Column (8). This rise was due to some leakage into the stand-pipe at the diverter hinge from splash in the wasteway.

It was known from many observations that there was a negligible leakage from the valve at the bottom of the stand-pipe with low heights of water, increasing to a small but appreciable quantity for the filled stand-pipe. Consequently, as the data show, at the end the diminution in the rate of rise represents the rate of fall (relatively to the beginning) due to leakage from the stand-pipe—in this case, 0.007 ft. per min. With no leakage at the start and 0.007 ft. per min. at the end, an average of 0.0035 ft. per min. is used for the $\frac{39}{60}$ min. of the volumetric measurement. Then to the last of the initial readings (here taken 1 min. before the diverter was dropped) is added 0.032 ft. to give the initial reading at the instant of diversion. To the first of the final readings (here taken 2 min. after the diverter was raised) is added the 0.002 ft. to offset leakage from the stand-pipe during the run; finally, $2 \times 0.025 = 0.050$ ft. is subtracted on account of the leakage into the stand-pipe during the 2 min. elapsed. In this example, the corrections are small, but with heads of more than 2 ft. they were considerable.

The values in Column (9) of Table 3 are taken from the calibration curve of the stand-pipe for the corrected initial and final water levels. This curve permitted readings of heights to 0.005 ft. and of volumes to 0.1 cu. ft. In Column (4) the times of day for diverter "in" and "out" are noted as read on an ordinary watch to the nearest second. The exact duration as scaled from the chronograph ribbon is also recorded. In Column (5) the readings of the weir float-gauge before, during, and after the run are shown. Column (6) gives the readings of the plumb-bob gauge before and after the run, and shows that no readings by this gauge were taken during Run D-55.

TABLE 3.—SAMPLES OF ORIGINAL NOTES, SHOWING OBSERVED DATA AND PRIMARY CALCULATIONS.

SERIES D WEIR, 7.5 FEET HIGH.

OBSERVED DATA.						COMPUTED DATA.			
Number of run.	Time of day.	Stand-pipe float-gauge, in feet.	Duration of volumetric measurement.	Weir float-gauge, in feet.	Weir plumb-bob gauge, in feet.	Weir gauge averages and head calculations, in feet.	Stand-pipe rates of fall or rise and adjusted initial and final readings.	Contents, in cubic feet, initial, final, and difference.	Discharge, Q, in cubic feet per second.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
55	10 : 31 : 00	4.633		2.511	27.136		Beginning		
	10 : 31 : 15	4.642		2.490	27.136	Float :	+ 0.032 ft. per min.	.....	.....
	10 : 31 : 30	4.651		2.506	27.130	2.4999	Ending		
	10 : 31 : 45	4.657		2.496	27.142	0.6949	+ 0.025 ft. per min.	.....	.....
	10 : 32 : 00	4.665		2.509	.....	1.8050	2| 0.007		
	.....	.....	In, 33 : 00	2.493	.....		0.0035	.....	.....
	.....	.....	Out, 33 : 40	2.500	.....	Plumb-bob :	$0.0035 \times \frac{39}{60} = 0.002$	.....	.....
	10 : 35 : 40	52.478	Chronograph	2.488	.....	27.1342	4.665	.....	.....
	10 : 35 : 55	52.485	shows 38.99 sec.	2.498	27.135	25.3249	+ 0.032	.....	.....
	10 : 36 : 10	52.492		2.506	27.133	1.8048	4.697 initial	125.6	.....
	10 : 36 : 25	52.498		2.497	27.129		52.478	.....	31.9364
	10 : 36 : 40	52.503		2.505	27.133		− 0.050	.....	.....
	.....	.....		.....	.....		+ 0.002	.....	.....
	.....	.....		.....	.....		52.430 final	1489.7	.....
	.....	.....		.....	.....			1364.1 Difference	.....

TABLE 3.—(*Continued.*)

SERIES J WEIR, 4.0 FEET HIGH.

OBSERVED DATA.						COMPUTED DATA.			
Number of run.	Time of day.	Stand-pipe float-gauge, in feet.	Duration of volumetric measurement.	Weir float-gauge, in feet.	Weir plumb-bob gauge, in feet.	Weir gauge averages and head calculations, in feet.	Stand-pipe rates of fall or rise and adjusted initial and final readings.	Contents, in cubic feet, initial, final, and difference.	Discharge, Q, in cubic feet per second.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
14	1:07:00	6.293		0.875	25.508	Float:	Beginning	.....	.....
	1:31:00	6.280		0.8745	25.508	0.8747	— 0.0005 ft. per min.	.....	.....
	.....	.....	In, 1:31:00	0.875	25.5075	0.6959	Ending	.....	.....
	.....	.....	Out, 1:41:00	0.8745	25.5075		— 0.0055 ft. per min.	.....	.....
	1:43:00	28.754	600 sec.	0.8745	25.508	0.1788		.....	.....
	1:45:00	28.748		0.875	25.508	Plumb-bob:	2|—0.006	.....	.....
	.....	.....		0.8745	25.508	25.5078	— 0.003	.....	.....
	.....	.....		0.8745	25.5075	25.3290	6.280 initial	170.4	.....
	.....	.....		.....	.....		28.754	.....	.....
	.....	.....		.....	.....	0.1788	+ 0.011	.....	1.0703
	.....	.....		.....	.....		+ 0.030	812.6	.....
	.....	.....		.....	.....		28.795 final	642.2	.....
	.....	.....		.....	.....			Difference	

Column (7) shows the average gauge readings, and the subtraction of the zero readings to obtain the heads on the weir crest. In Series D it was decided to average all the weir float-gauge readings rather than to use only those taken during the run, although this latter procedure was the usual one when sufficient readings were available. In this case there were five consecutive runs, Nos. 51 to 55, inclusive, at the same valve setting. In only three runs, Nos. 51, 52, and 55, was the float-gauge read during the run, but in all of them readings were taken just before and just after the run, as was the case also for the plumb-bob gauge. The readings during Runs Nos. 51, 52, and 55 by float-gauge as compared to the average of readings before, during, and after the test, showed a deficiency of 0.0009 ft., or 1 part in 2 000; as compared to the grand average by both gauges, the deficiency was only 0.0004 ft. The average head by float-gauge in Runs Nos. 53 and 54 (readings taken only before and after the run) shows exact agreement (h = 1.8039 ft.) with the grand average.

In Run No. 14, Table 3, covering Series J (typical low head), all the weir head readings were taken during the volumetric measurement, which lasted 10 min. The duration was controlled by an ordinary well-regulated watch, the observer moving the diverter to the nearest 0.2 sec. The steadiness of the water surface in the channel of approach tempted the observer to estimate 0.0005 ft. in reading the gauges.

For the very highest heads the leakage into the stand-pipe from the waste-way before and after the measurement was greater than in Run No. D-55. For heads of 2.0, 2.25, 2.50, and 2.75 ft. in Series D the rates of rise in the stand-pipe were, respectively, 0.075, 0.45, 1.5, and 2.5 ft. per min., so that great care was used in noting the exact second the diverter went in and out. Also, at least five readings on the stand-pipe gauge were taken at exact 15-sec. intervals before and after the measurement, in order to determine the rate of rise closely. The weir-head gauges were each read, usually six times, immediately before and, again, immediately after the measurement.

Velocity Measurements

Velocity measurements in the channel of approach were made for the relatively higher heads, just before or just after the volumetric measurements, using a small Price electric current meter with a telephone receiver. The meter was held in position by an iron rod graduated to feet and tenths, and was operated through the same cross-section in which the head was measured. The meter was rated several times during the course of the experiments and found to remain substantially constant in its indications.

Velocities.—For most of the writers' Series A to P, only the velocities along a single vertical element at mid-stream were observed. For Series F, velocities 0.5 ft. from either side-wall were read for a head of 2.36 ft. on the weir. For Series K, velocities in five verticals were observed at 2-ft. head. Fig. 42 shows the horizontal distribution of velocities. Appendix B tabulates the results of all velocity measurements, and Figs. 33 to 47 show the curves of velocity distribution.

For Series N (0.75-ft. weir) and Series K (5.5-ft. weir) additional vertical mid-stream traverses were made 6 ft. up stream from the weir to determine whether any considerable re-arrangement of velocities had been effected as the flow approached the weir between the sections, 11.74 ft. and 6 ft. up stream. Figs. 38 and 43 show the comparative distributions. Little change is seen for the 2-ft. head of Series K; but in three of the four cases of Series N there is an indication of a slowing up of the mid-channel velocities, probably at the expense of increasing wall velocities.

With a weir, 3.76 ft. high and 5 ft. long, and with screens 17.7 ft. up stream from the weir, Fteley and Stearns made three velocity measurements for a head of 0.78 ft., in each of three verticals at 16 ft. and also at 8 ft., up stream from the weir. They conclude* that "8 ft. is not a sufficient distance to cause even a partial re-arrangement, such as might be looked for."

Fig. 48 shows a velocity distribution measured by Fteley and Stearns 5 ft. up stream from their 6.55-ft. weir, 19 ft. long, with a head of 1.304 ft.† This measurement stands alone as a definite velocity traverse showing channel conditions in the classic experiments of Francis, Fteley and Stearns, and Bazin.

Adjustment of Velocities.—The average head on the weir during a set of current-meter measurements usually differed slightly from the head during the corresponding volumetric measurements (Appendix B), so that, to obtain the discharge during the current-meter measurements, the observed discharge by volumetric measurement needed to be corrected slightly, according to the law of variation between Q and h for the particular weir at the particular head. (For the high weirs, Q varies nearly as $h^{\frac{3}{2}}$. For the lower weirs at the high heads, the exponent of h is appreciably more than $\frac{3}{2}$, being 1.85 in Series E. It was found for each weir by logarithmic plotting of the values actually measured.)

The true mean velocity of approach prevailing during the current-meter measurements was then calculated by dividing the adjusted volumetric discharge per foot of width by the depth of water in the channel of approach. In general, the mean velocity as found by current meter (above a single vertical element) differed from this true mean velocity by volumetric measurement. The ratio was computed and assumed to apply to each of the component parts of the curve of velocity distribution in the vertical, such as V_a, the mean velocity above the crest level, and V_b, the mean velocity below the crest level.

In a few cases (see Appendix B (Table 55)), the range of velocities was so extreme that the mean of the velocity heads is sensibly in excess of the head corresponding to the mean velocity. In such cases both values are shown, but the mean of the velocity heads is used in subsequent computations.

New Experimental Data by Others

Other experimental data on weirs without end contraction, herein published for the first time, are parts of the results of experiments made at Cornell University by Messrs. Charles L. Walker and Carl R. Weidner (1904)

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 38.
† *Loc. cit.*, p. 74.

while Senior students; and the following men while Graduate Students in Civil Engineering: William F. Martin (1906); Francis M. Dawson (1913); Henry R. J. Meyer and Russell A. See (1914); Jacob O. Jones (1915); and Charles M. Weber (1915).* All these experiments were on sharp crests except the rounded crests of Dawson and Jones.

The writers by personal association had intimate knowledge of the details of all these experiments. The first mentioned experimenters did their work under Gardner S. Williams, M. Am. Soc. C. E., in charge of Experimental Hydraulics at Cornell University. All the others worked with one of the writers in charge.

Walker and Weidner's Experiments.—These experiments showed the effects on the discharge caused by the width (thickness) of the crest piece ($\frac{1}{64}$ in. to 2 in.) at low heads (0.02 to 0.13 ft.). They were made on a weir 0.6 ft. high, set in a wooden flume, 0.895 ft. wide and 12 ft. long. The weir bulkheads for the $\frac{1}{64}$-in. and the $\frac{1}{8}$-in. thicknesses consisted of brass plates, and the $\frac{1}{4}$-in. crest was the edge of a varnished cherry wood plate set against the brass bulkhead with the $\frac{1}{64}$-in. crest. The water supply was by gravity at low velocity through a 6-in. pipe from an auxiliary low-level tank fed by a 2-in. pipe from a tank 45 ft. above, the water level of which was kept nearly constant by a float-valve. Heads were measured by a hook-gauge in the channel of approach 1.58 ft. up stream from the weir. The scale was graduated to 0.0025 ft. and a vernier permitted readings to 0.0001 ft. Discharges were measured by diverting established flows into a weighing tank of 2000 lb. capacity. Periods of time were measured with an ordinary watch. The data are presented on Figs. 26 and 29, but are not tabulated.

Martin's Experiments.—Martin made his experiments to study the effects on discharge caused by the width of the channel of approach ($1\frac{1}{2}$ in. to 2 ft.). Only the data for the weir in his widest flume, 2 ft., are given here (Appendix A (Table 30) and Fig. 49). The weir was 1.2 ft. high with a double set of baffles, 10 ft. up stream. The crest piece was a brass plate, $\frac{1}{16}$ in. thick, its edge milled to a straight-edge, set flush with the up-stream face of the weir bulkhead. The water supply was by gravity, from a 12-in. connection at the bottom of the 6-ft. stand-pipe already described. At this connection there was a 12-in. gate-valve and an up-turned elbow feeding a 12-in. riser pipe, 12 ft. long, terminating in the bottom of the flume 2.5 ft. from the up-stream end.

Heads were measured 7.25 ft. up stream from the weir by two gauges: (1) A plumb-bob gauge for direct readings of the water surface in the flume; and (2) a piezometer consisting of a 1-in. hole through the side wall near the bottom. Into this hole a $\frac{3}{4}$-in. pipe was screwed from the outside so as to be exactly flush on the inside. The pipe led into the bottom of a cylindrical glass jar, 8 in. in diameter and 2 ft. high, used as a stilling-well, in which the water surface was read by a point-gauge. The given heads are the average for the two gauges, on each of which three to six separate readings were made during the diversion into the tank.

Discharges were measured by diverting the established flows into a lead-lined, 10 by 16-ft. measuring tank, 5 ft. deep, calibrated by withdrawing suc-

* Messrs. Walker, Weidner, Meyer, Jones, and Weber are now members of the Society.

cessive 5-cu. ft. batches (by weight). Depths of water in the tank were measured by two plumb-bob gauges. All durations were measured both by a good stop-watch and by an ordinary watch read to 0.2 sec.

Dawson's Experiments.—These experiments relating to effects on discharge caused by small roundings of the up-stream top corner of the crest piece, covered sharp and nearly sharp crests and roundings to radii ranging from $\frac{1}{8}$ in. to $1\frac{1}{2}$ in. (Fig. 3). The same flume, apparatus, and methods were used as in the writers' experiments. Only the data for the sharp and nearly sharp crests are given in Appendix A (Tables 25 to 27), being plotted on Figs. 8 and 49, but the data for some of the roundings are represented on Figs. 27, 28, and 29.

Meyer and See's Experiments.—Meyer and See investigated the effects on discharge caused by the height of the weir (covering somewhat the same ground as Bazin's volumetric experiments). These tests were made in a concrete flume* (Fig. 7) 2 ft. wide, 4 ft. deep, and 30 ft. long. There were two sets of experiments. In Set I, the weir bulkhead was progressively lowered, the fixed concrete bottom of the flume being the bottom of the channels of approach. In Set II, the weir bulkhead remained 2.5 ft. high, as in Series A of Set I, but an auxiliary movable wooden bottom (Fig. 7) was introduced and lowered progressively from near crest level to 1.8 ft. below crest level. The water supply was by gravity from Beebe Lake through a 10-in. pipe leading from the 30-in. pipe.

Heads were measured by two float-gauges each connected, in Set I, to one of opposite piezometer holes formed by 1-in. pipes extending through the concrete sides of the flume close to the bottom. The float-pipes and floats were of the type used in the writers' experiments. In Set II, the piezometer holes were in the wooden bottom and a flexible connection was secured by using short lengths of $\frac{3}{4}$-in. rubber hose between $\frac{1}{2}$-in. pipe nipples screwed into the wooden bottom from below and similar nipples cemented into the piezometer holes used in Set I. The piezometer holes of Set II were located 6 in. on either side of the center line of the flume, the same distance from the weir as in Set I.

Discharges were measured by diversion of the established flow into one of two tanks, either a cylindrical tank of galvanized sheet iron of 70 cu. ft. capacity, or a rectangular concrete tank of 400 cu. ft. capacity. The discharge from the weirs fell into a large hopper funnel (Fig. 7) forming the upper end of a down-spout free to swing at its lower end thus turning the water to waste or into one of the tanks.

Durations of time were measured by an electric chronograph substantially as described for the writers' experiments. Velocities in the channel of approach were measured for only a few cases, in Set I, namely, for three heads each in Series C, E, and G. The data are given in Appendix B (Table 55) and plotted on Fig 47.

Jones' Experiments.†—These tests regarding effects on discharge caused by extremely small roundings of the up-stream top corner of the crest piece

*The flume and equipment were briefly described in *Engineering News-Record*, September 2, 1920, p. 452.

† A synopsis of Jones' results appeared in the *Cornell Civil Engineer*, November, 1917.

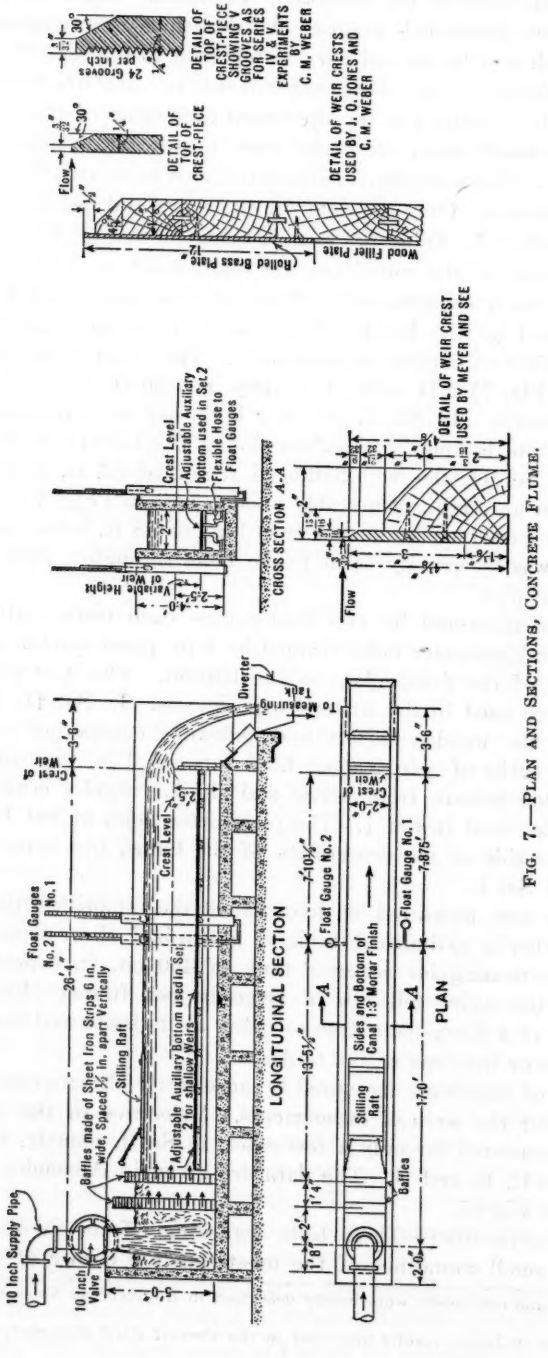
DETAIL OF TOP OF CREST-PIECE SHOWING V GROOVES AS FOR SERIES IV & V EXPERIMENTS BY C. M. WEBER
DETAIL OF TOP OF CREST-PIECE
DETAIL OF WEIR CRESTS USED BY J. O. JONES AND C. M. WEBER
Flow
Wood Filler Plate
(Rolled Brass Plate 12"
DETAIL OF WEIR CREST USED BY MEYER AND SEE
Crest Level
Adjustable Auxiliary Bottom used in Set.2
Flexible Hose to Float Gauges
Variable Height of Weir
CROSS SECTION A-A
Diverter
To Measuring Tank
Crest of Weir
Float Gauges No. 1
No. 2
Crest Level
Adjustable Auxiliary Bottom used in Set. 2 for Shallow Weirs
Baffles made of Sheet Iron Strips 6 in. wide, Spaced 5½ in. apart Vertically
Stilling Raft
10 Inch Supply Pipe
10 Inch Valve
LONGITUDINAL SECTION
Float Gauge No. 2
Float Gauge No. 1
Sides and Bottom of Canal 1:3 Mortar Finish
Stilling Raft
Baffles
PLAN

FIG. 7.—PLAN AND SECTIONS, CONCRETE FLUME.

were made with the same flume and measuring devices as used by Meyer and See (Fig. 7), with a weir 2.5 ft. high. The crest piece was a rolled brass plate, $\frac{1}{4}$ in. by 12 in., and 2 ft. 0 $\frac{3}{16}$ in. long—other details as shown on Fig. 7. At first, the up-stream top corner was square and sharp. Then, progressively, the edge was "broken" and approximately rounded more and more for the succeeding series of experiments by light rubbings with a fine emery cloth. To measure the degree of rounding, impressions were taken with dental wax, and a section of the wax was micro-photographed. The data are given in Appendix A (Tables 42 to 49, inclusive) and on Fig. 30. Velocity measurements are given in Appendix B (Table 55), but the data are not plotted.

Weber's Experiments.—The effects on discharge due to the degree of roughness of the up-stream face of the weir near the crest, were made by Weber in conjunction with Jones, using the same height of weir and type of crest piece. The roughness was made by cutting a series of adjoining 60° V-grooves horizontally across the crest plate, starting in Series II, with 96 grooves per in. for 4 in. vertically below the crest, leaving the up-stream top corner square and sharp (Fig. 7). This kept the summits between the grooves in the same plane as for the smooth plate, and avoided a possible criticism of Barr's method of adding shellac and sand to the face.

The next degree of roughness, for Series III, was obtained by cutting 48 grooves per in., removing every other one of the smaller ridges; then, for Series IV, 24 grooves per in. were cut; and, finally, for Series V, as a modification of this last condition, the bottom 8-in. part of the brass plate, hitherto left smooth, was roughened by cutting 15 grooves per in. Series VI reproduced the conditions of Series III, 48 grooves per in. for the upper 4 in. of a 12-in. plate, with the lower 8 in. smooth. Weber's data are given in Appendix A (Tables 50 to 54, inclusive) and on Fig. 32. The velocity measurements are given in Appendix B (Table 55), but are not plotted.

Discussion on the New Data

General

In Appendix A (Tables 9 to 24, inclusive) are given measured heads and discharges per foot of length of weir for the writers' weirs; and also the percentage departures of the measured discharges from those computed for the same head by the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}}$ (in foot-second units), or $Q = \frac{2}{3} \times 0.623\ L\ h\ \sqrt{2\ g\ h}$.

The latter part of Appendix A (Tables 25 to 54, inclusive) gives corresponding data for the experiments by Walker and Weidner, Martin, Dawson, Meyer and See, Jones, and Weber.

Figs. 8 to 13 and Figs. 23 to 29, inclusive, and Fig. 49, are plottings of the percentage departures with auxiliary scales of coefficients. This method of plotting places the data a little more definitely on a precision basis, but is really only a slight departure from the type of diagrams used by Fteley and Stearns, Bazin, and Rehbock, in showing variations of coefficients with the heads for various heights of weir.

LEGEND
Series A, Schoder and Turner
Series B, " "
Series I, Dawson
Series II, Dawson
Series III, Dawson
Series C, Schoder and Turner
For Low Heads See Fig. 23, 28 & 29
No. 1
No. 2
No. 3
No. 4
Series I, Dawson
Series A, S. and T.
Series C, S. and T.
Series III, Dawson
Series B, S. and T.
Series II, Dawson
Observed Head (h) in Feet
Percentage Variation of Measured Discharge from Discharge by Francis Formula Q = 3.33 h^3/2
Coefficient "C" in Formula, Q = Ch^3/2 per Foot Length of Weir

FIG. 8.—COMPARISON OF FLOW OVER SHARP OR NEARLY SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS. SIMILAR CONDITIONS FOR TESTS EXCEPT SLIGHT VARIATIONS IN SHARPNESS OF UP-STREAM TOP CORNER OF CREST, SMOOTHNESS OF UP-STREAM FACE, AND VERTICAL DISTRIBUTION OF VELOCITIES IN CHANNEL OF APPROACH.

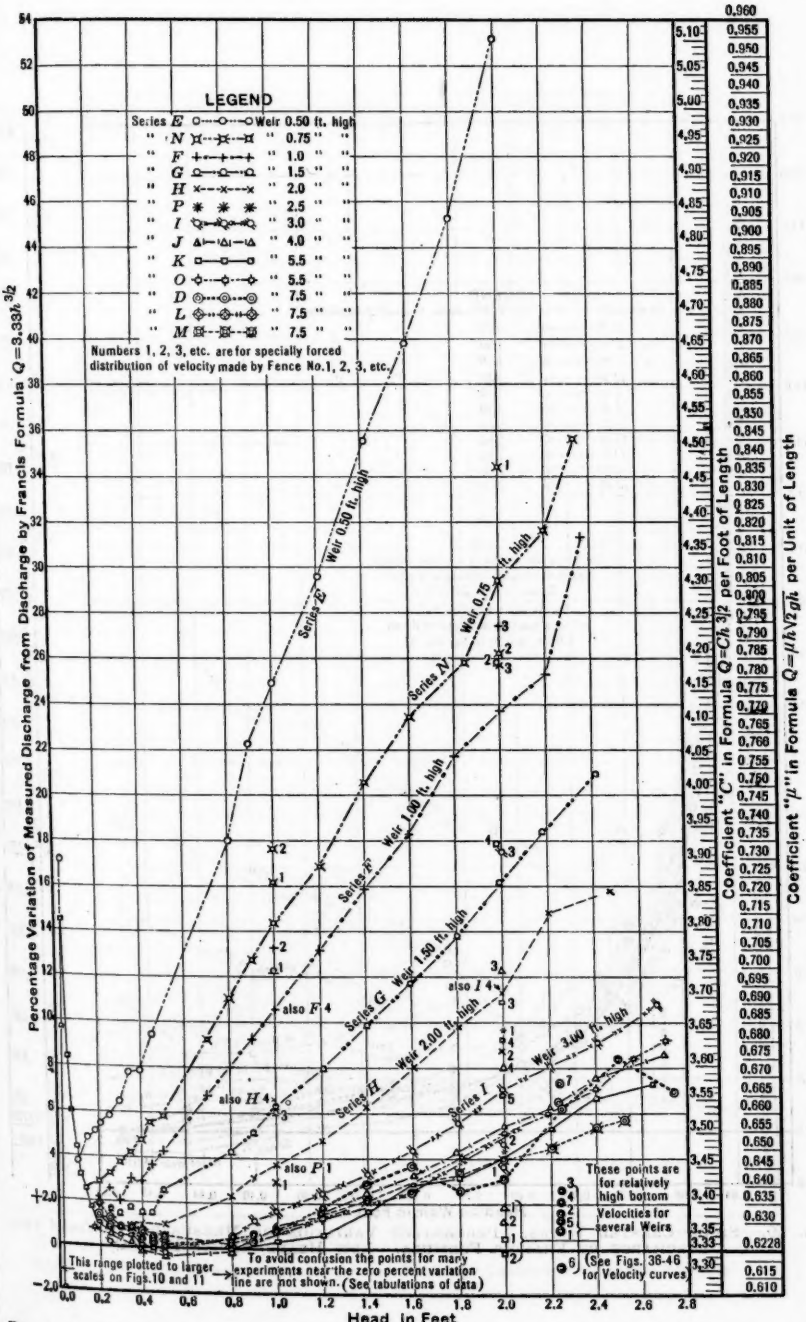
LEGEND
Series E Weir 0.50 ft. high
" N " 0.75 " "
" F " 1.0 " "
" G " 1.5 " "
" H " 2.0 " "
" P " 2.5 " "
" I " 3.0 " "
" J " 4.0 " "
" K " 5.5 " "
" O " 5.5 " "
" D " 7.5 " "
" L " 7.5 " "
" M " 7.5 " "
Numbers 1, 2, 3, etc. are for specially forced distribution of velocity made by Fence No.1, 2, 3, etc.
Series E Weir 0.50 ft. high
Series N Weir 0.75 ft. high
Series F Weir 1.00 ft. high
Series G Weir 1.50 ft. high
Series H Weir 2.00 ft. high
Series I Weir 3.00 ft. high
also F 4
also H 4
also I 4
also P 1
These points are for relatively high bottom Velocities for several Weirs
This range plotted to larger scales on Figs.10 and 11
To avoid confusion the points for many experiments near the zero percent variation line are not shown. (See tabulations of data)
(See Figs. 38-46 for Velocity curves)
Percentage Variation of Measured Discharge from Discharge by Francis Formula $Q=3.33h^{3/2}$
Head in Feet
Coefficient "C" in Formula $Q=Ch^{3/2}$ per Foot of Length
Coefficient "μ" in Formula $Q=\mu lh\sqrt{2gh}$ per Unit of Length

FIG. 9.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA, SERIES D TO P.

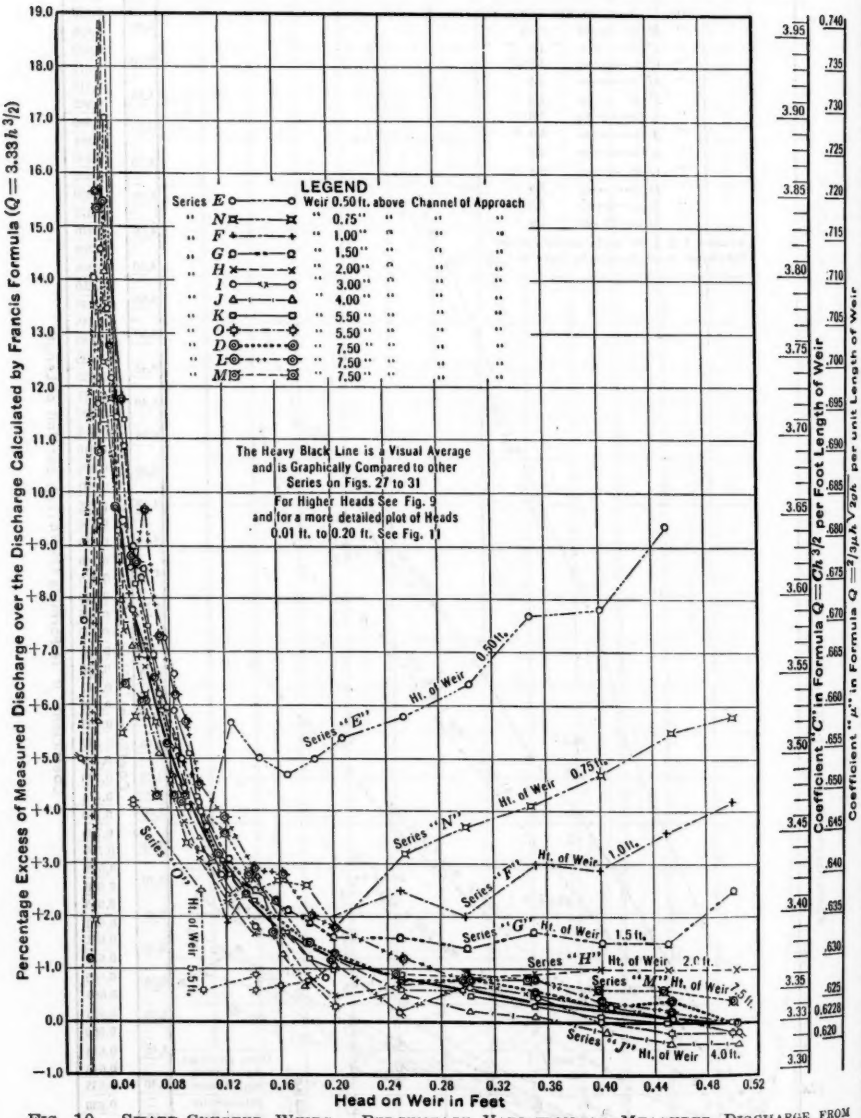
LEGEND
Series E Weir 0.50 ft. above Channel of Approach
" N " 0.75 " " " "
" F " 1.00 " " " "
" G " 1.50 " " " "
" H " 2.00 " " " "
" I " 3.00 " " " "
" J " 4.00 " " " "
" K " 5.50 " " " "
" O " 5.50 " " " "
" D " 7.50 " " " "
" L " 7.50 " " " "
" M " 7.50 " " " "
The Heavy Black Line is a Visual Average and is Graphically Compared to other Series on Figs. 27 to 31
For Higher Heads See Fig. 9 and for a more detailed plot of Heads 0.01 ft. to 0.20 ft. See Fig. 11
Percentage Excess of Measured Discharge over the Discharge Calculated by Francis Formula ($Q = 3.33h^{3/2}$)
Head on Weir in Feet
Coefficient "C" in Formula $Q = Ch^{3/2}$ per Foot Length of Weir
Coefficient "μ" in Formula $Q = \frac{2}{3}\mu k\sqrt{2g}$ per Unit Length of Weir
Series "E" Ht. of Weir 0.50 ft.
Series "N" Ht. of Weir 0.75 ft.
Series "F" Ht. of Weir 1.0 ft.
Series "G" Ht. of Weir 1.5 ft.
Series "H" Ht. of Weir 2.0 ft.
Series "M" Ht. of Weir 7.5 ft.
Series "J" Ht. of Weir 4.0 ft.
Series "O" Ht. of Weir 5.5 ft.

FIG. 10.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA. LOW HEADS, SERIES D TO O.

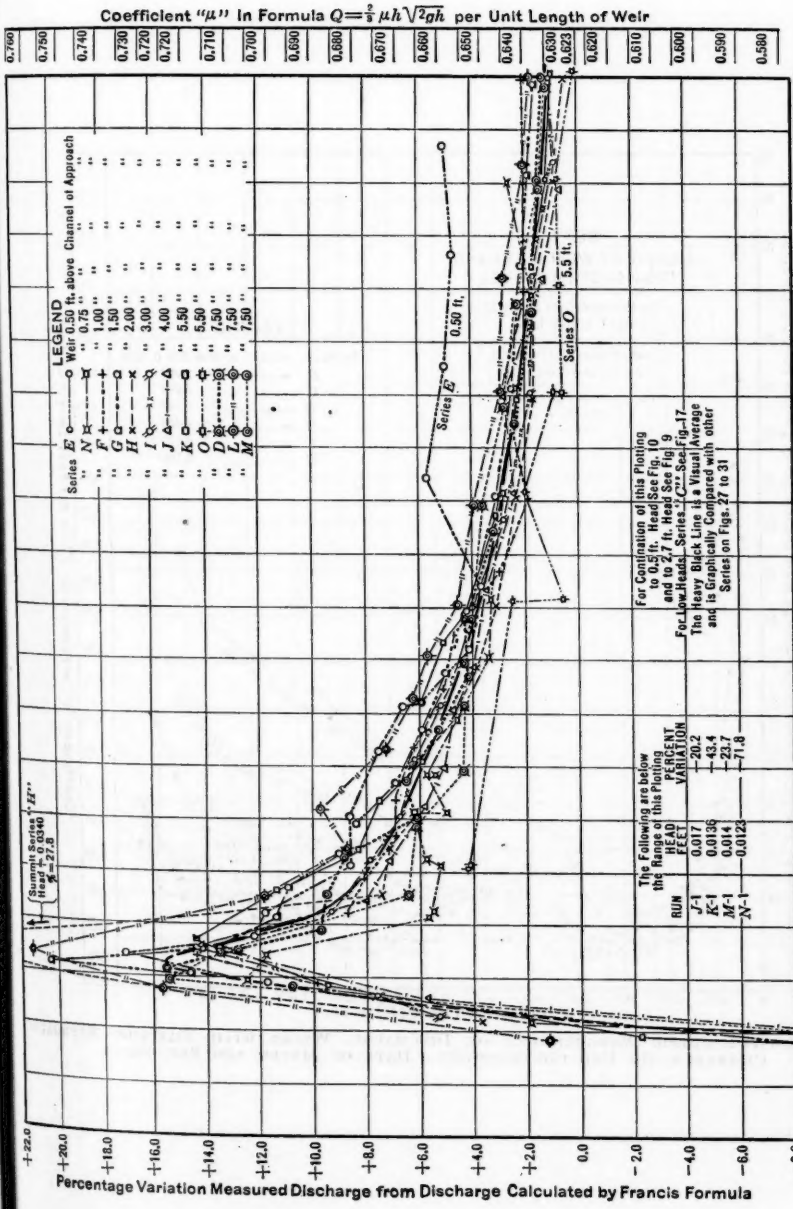

FIG. 11.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA. EXTREMELY LOW HEADS, SERIES D TO N.

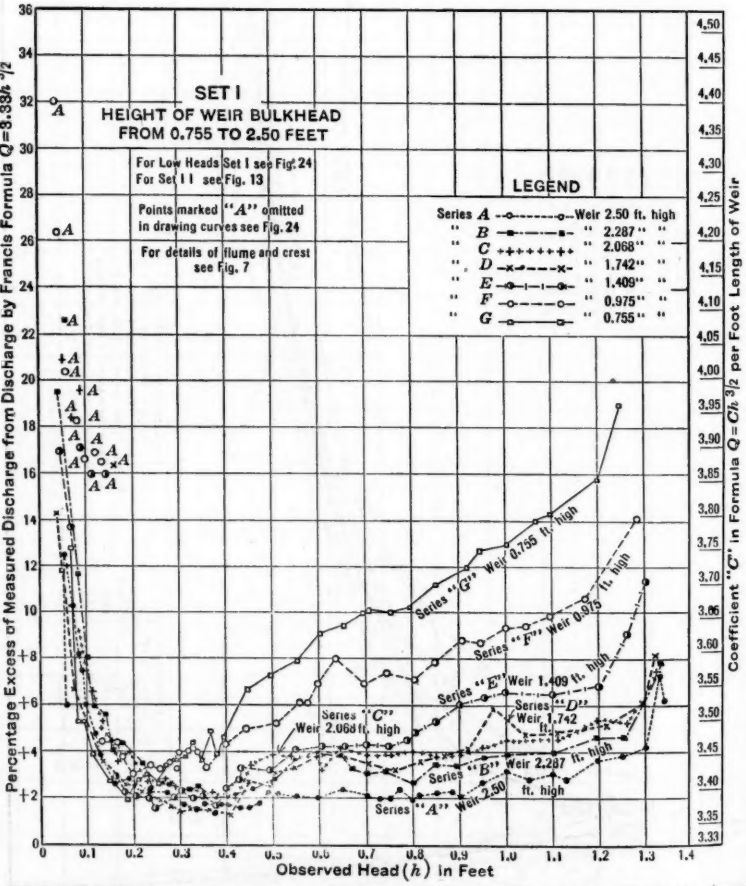
SET I
HEIGHT OF WEIR BULKHEAD
FROM 0.755 TO 2.50 FEET
For Low Heads Set I see Fig. 24
For Set II see Fig. 13
Points marked "A" omitted in drawing curves see Fig. 24
For details of flume and crest see Fig. 7
LEGEND
Series A Weir 2.50 ft. high
" B " 2.287 " "
" C " 2.068 " "
" D " 1.742 " "
" E " 1.409 " "
" F " 0.975 " "
" G " 0.755 " "
Percentage Excess of Measured Discharge from Discharge by Francis Formula $Q=3.33h^{3/2}$
Coefficient "C" in Formula $Q=Ch^{3/2}$ per Foot Length of Weir
Observed Head (h) in Feet
Series "G" Weir 0.755 ft. high
Series "F" Weir 0.975 ft. high
Series "E" Weir 1.409 ft. high
Series "D" Weir 1.742 ft. high
Series "C" Weir 2.068 ft. high
Series "B" Weir 2.287 ft. high
Series "A" Weir 2.50 ft. high

FIG. 12.—VOLUMETRIC MEASUREMENT OF DISCHARGE, WEIRS WITH SHALLOW APPROACH CHANNELS, NO END CONTRACTIONS. DATA OF MEYER AND SEE, SET I.

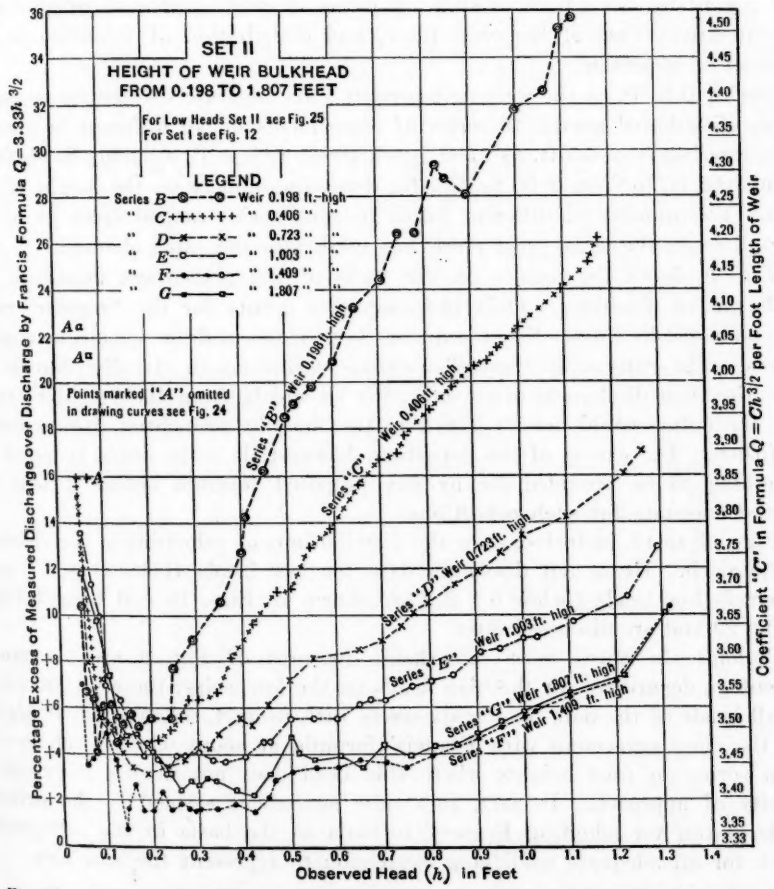
SET II
HEIGHT OF WEIR BULKHEAD
FROM 0.198 TO 1.807 FEET
For Low Heads Set II see Fig. 25
For Set I see Fig. 12
LEGEND
Series B — Weir 0.198 ft. high
" C " " 0.406 " "
" D " " 0.723 " "
" E " " 1.003 " "
" F " " 1.409 " "
" G " " 1.807 " "
Points marked "A" omitted in drawing curves see Fig. 24
Series "B" Weir 0.198 ft. high
Series "C" Weir 0.406 ft. high
Series "D" Weir 0.723 ft. high
Series "E" Weir 1.003 ft. high
Series "G" Weir 1.807 ft. high
Series "F" Weir 1.409 ft. high
Percentage Excess of Measured Discharge over Discharge by Francis Formula $Q=3.33h^{3/2}$
Coefficient "C" in Formula $Q=Ch^{3/2}$ per Foot Length of Weir
Observed Head (h) in Feet

FIG. 13.—VOLUMETRIC MEASUREMENT OF DISCHARGE, WEIRS WITH SHALLOW APPROACH CHANNELS, NO END CONTRACTIONS. DATA OF MEYER AND SEE, SET II.

Francis' formula, as a tentative reference datum, was used for convenience in computations because of the availability of excellent tables* of discharges by this formula for heads differing by 0.001 ft. (interpolation to 0.0001 ft.).

New Data.—The writers' results yield thought-compelling illustrations of the magnitude of percentage variations in discharge. Particularly is this true if one adopts, for the moment, the usual custom of taking quantitative account only of the head of water and height of weir, with little or no definitely specified regard for the effects of slight dullness of crest, degree of roughness of the up-stream face of the crest piece, and distribution of velocities in the channel of approach.

Series D to P, by the writers, represent 1 017 separate volumetric measurements distributed among 13 series of experiments on 10 different heights of weir, ranging from 0.5 ft. to 7.5 ft., each series, except P, covering heads from about 0.02 ft. to from 2 ft. to 2.75 ft., depending chiefly on the height of the weir. The number of different heads in each series ranged from 24 to 42. For all weirs the same crest piece was used, also the same channel.

Fig. 9 shows the results on the basis of the percentage departure, or coefficient of discharge. Only the successive points for the "regular" runs are connected by lines. No attempt has been made to draw smooth averaging curves. The runs with "fences", causing variations in the distribution of velocities, had discharges often departing so widely from the "regular" runs that the points could not be joined to the lines by branching lines without confusion. The extent of the departure, however, is to be borne in mind as something to be provided for by any proposed formula before it may be deemed adequate for such conditions.

Figs. 33 to 46, inclusive, show the distributions of velocities in the channel of approach. Head and discharge data for low heads (below 0.5 ft.) and extremely low heads (below 0.2 ft.) are shown in Figs. 10 and 11 and Figs. 28 and 29 and are discussed later.

Perhaps the three most remarkable features of Fig. 9 are the great percentage departures for the high heads on the low weirs; the close grouping for all heads of the data on the six series with weirs 4, 5.5, and 7.5 ft. high; and the close agreement with Francis' formula at about 0.5-ft. head for all seven series on four heights where this head does not involve appreciable velocity of approach. In fact, this close agreement seemed to the writers ample reason for adopting Francis' formula as the basis in the subsequent search for an adequate modifying expression to represent the new data.

Old Formulas Tested by New Measurements

Figs. 14, 15, and 16 show in detail to what extent the discharges calculated for the measured heads by the formulas of Fteley and Stearns and Bazin, and the weir curves of Lyman, fail to agree with the measured discharges of the writers' "regular" runs of Series A to P. It is obvious that all these old formulas fail to fit even the general trend of the writers' "regular" runs.

* The tables by R. E. Horton, M. Am. Soc. C. E., in *Water Supply and Irrigation Paper No. 200*, U. S. Geological Survey, were used by the writers

FIG. 14.—PLOTTING TO TEST FTELEY AND STEARNS' FORMULA BY MEASURED VALUES FOR SHARP-CRESTED WEIRS.

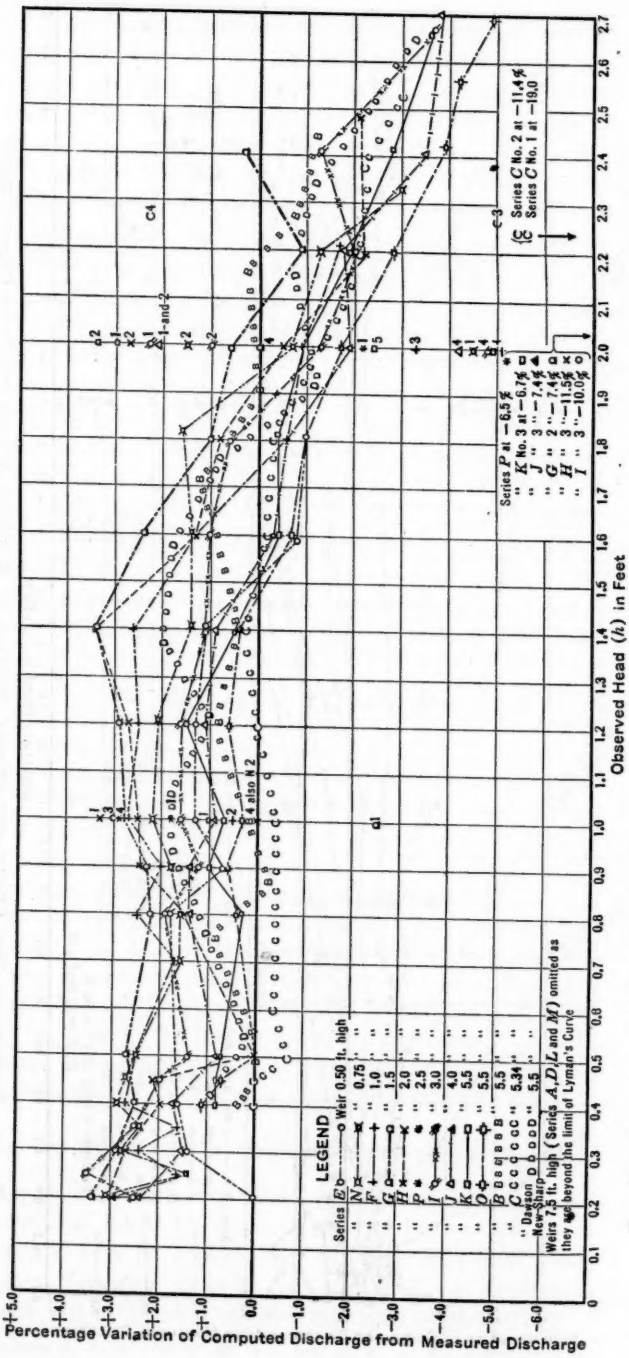
Observed Head (h) in Feet
Percentage Variation of Computed Discharge from Measured Discharge
LEGEND
Series E Weir 0.50 ft. high
" N " " 0.75 " "
" F " " 1.0 " "
" G " " 1.5 " "
" H " " 2.0 " "
" P " " 2.5 " "
" I " " 3.0 " "
" J " " 4.0 " "
" K " " 5.5 " "
" Q " " 5.5 " "
" B " " 5.5 " "
" C " " 5.34 " "
" Dawson D " " 5.5 " "
New Sharp Weirs 1.5 ft. high (Series A, D, L and M) omitted as they lie beyond the limit of Lyman's Curve
Series P at —6.5%
" K No. 3 at —6.7%
" J " 3 " —7.4%
" G " 2 " —7.4%
" H " 3 " —11.5%
" I " 3 " —10.0%
Series C No. 2 at —11.4%
Series C No. 1 at —19.0%

FIG. 15.—PLOTTING TO TEST LYMAN'S WEIR DISCHARGE CURVES, BY MEASURED VALUES, FOR SHARP-CRESTED WEIRS.

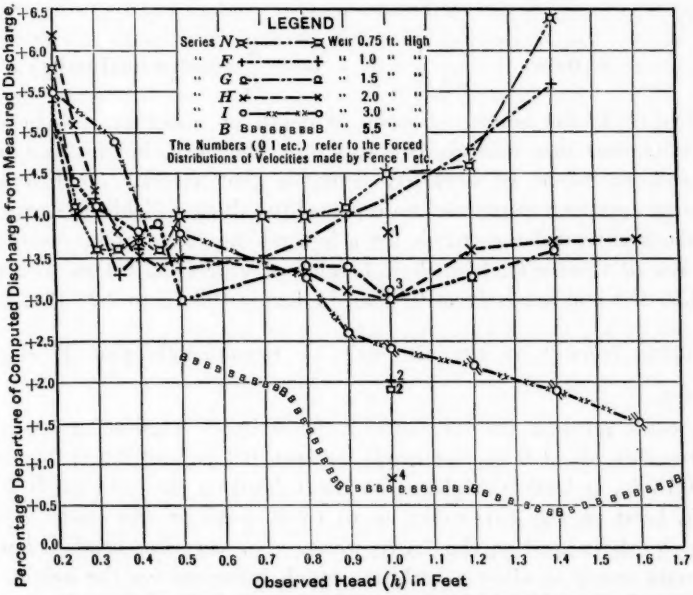
LEGEND
Series N Weir 0.75 ft. High
" F " 1.0 " "
" G " 1.5 " "
" H " 2.0 " "
" I " 3.0 " "
" B " 5.5 " "
The Numbers (0 1 etc.) refer to the Forced Distributions of Velocities made by Fence 1 etc.
Percentage Departure of Computed Discharge from Measured Discharge
Observed Head (h) in Feet

FIG. 16.—PLOTTING TO TEST BAZIN'S WEIR COEFFICIENT BY MEASURED VALUES FOR SHARP-CRESTED WEIRS.

Fteley and Stearns' Formula.—This formula fails entirely to fit the four lowest weirs, giving much too high discharges. Above heads of 1.2 ft. the formula gives too low discharges for the highest weirs.

Bazin's Coefficients (and Formula Approximating Them).—These not only yield too high discharges within the range of his own experiments, but show rapid divergences for heads between 1.0 and 1.5 ft. It was deemed unfair to apply Bazin's coefficients and formula to the writers' highest heads on the lowest weirs, because Bazin practically avoided the data of his own lowest weir, 0.79 ft. high, and stated that a very low weir should always be avoided. Comparisons at low heads with Bazin's data appear on Fig. 28.

*Lyman's Averaging Weir Curves.**—Lyman's curves give discharges in general too large for the low heads and too small for the high heads. They are quite inadequate for the cases where the velocity distribution in the channel of approach differed considerably from the "regular" runs.

Rehbock's Formula.—Apparently, this formula, almost unknown in the United States, was published in Germany in 1911. Rehbock's experiments were made on sharp-crested, fully aerated weirs, without end contractions, of heights of $\frac{1}{8}$, $\frac{1}{8}$, $\frac{1}{4}$, and $\frac{1}{2}$ m. (0.41, 0.66, 0.82, and 1.64 ft.) in a canal $\frac{1}{2}$ m. (1.64 ft.) wide with heads from 0.01 to 0.18 and 0.23 m. (0.03 to 0.59 and 0.75 ft.).

In 1912 he published† his revised formula for μ in the equation,

$$Q = \frac{2}{3} \mu L h \sqrt{2 g h}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), Plate XXI, p. 1195.

† Handbuch der Ingenieur Wissenschaften, III Teil, Wasserbau, Band 2 (Stauwerke), Vierte Auflage, 1912, p. 59.

namely,

$$\mu = 0.605 + \frac{1}{320\,h - 3} + 0.08\,\frac{h}{d_0} \text{ (foot-second units)}$$

in which, d_0 is the height of weir, or depth of water at zero head. It is noteworthy that this formula contains only the head, height, and length of weirs and, therefore, of necessity demands that velocity conditions in the channel of approach duplicate those prevailing during Rehbock's experiments.

Rehbock extended the curves for μ beyond the range of his measurements by the law of similarity, to include heights of weir from 0.1 m. to 2 m. (0.33 ft. to 6.56 ft.) and heads from 0.01 m. to 1 m. (0.03 ft. to 3.28 ft.), the ratio, $\frac{h}{d_0}$, ranging from 0 to 1.0, although he recommends that it should not exceed 0.8.

Rehbock's formula fits the data of the writers' "regular" runs on Series D to O within about 0.5% for heads between 0.2 ft. and 2.0 ft. on the weirs 0.5 ft., 0.75 ft., and 1.0 ft. high; up to 1.5-ft. head on the weir, 1.5 ft. high; up to 1.8-ft. head on the 3-ft. weir; up to 1.2-ft. head on the 5.5-ft. weirs; but only up to 0.4-ft. head on the 7.5-ft. weirs. For very low heads below 0.2 ft. his formula seems to allow a little too much influence for the height of weir to agree with the general average of all the writers' weirs.

For heads of more than 1.5 ft. on the weirs 1.5 to 7.5 ft. high, Rehbock's formula gives from 1 to 4% less discharge than the writers' measurements for the "regular" runs, the largest departures being for the highest weirs. It gives generally less discharge (up to 2%) than the data of Fteley and Stearns, Martin, and Meyer and See.

Considering, however, only those runs of the writers' tests with "fences" of such a construction as to cause a nearly uniform distribution of velocities from surface to bottom of the channel of approach (omitting most of the runs with highest heads on the highest weirs) Rehbock's formula gives closer agreement.

Comments.—Of course, if the same sort of limitations be imposed the older formulas would also appear to better advantage. This brings up the matter of choice (where choice is possible in using a weir or dam for precise measurements of discharge) between (1) making velocity conditions in the channel of approach agree with the specified conditions for which a particular formula applies; and (2) making the formula suitable for such variations in velocity conditions as may actually occur between the numerous different cases that arise in practice.

One phase of the situation is illustrated by the actual conditions of velocity distribution and discharge in the writers' Series A to P for which a fixed baffle system was used, the same for all heads on any one weir, and with seemingly small changes between the different weirs. These changes, if any, were not in the main baffles shown on Fig. 1, but possibly in the angle made by small fender boards fastened between the main baffle-boards in order to bring about a distribution of velocities approximately symmetrical across the canal.

Another phase of the situation is illustrated by the differences in velocity conditions at the same head between different channels containing weirs of the same height. In some cases direct measurements were made; namely, the channels of Meyer and See, and of Jones, and those of the writers (Figs. 33 to 48). In other cases the indications of differences are given by the various trends of the coefficient curves at the high heads; namely, the data of Bazin, Fteley and Stearns, Martin, Meyer and See, the writers, etc. (See Fig. 49.)

The practical question would seem to be "to what extent is it possible to build a weir and its channel of approach of such dimensions, and with such a baffle system and stilling-raft, as to reproduce the velocity conditions implied for a formula that takes no account of differences in velocity distribution?" The data in this paper should be of assistance to the hydraulic engineer in answering this question.

Search for a Comprehensive Formula

In spite of the extreme conditions caused on the one hand by the low weirs with heads of water higher than the height of the weir, and on the other hand, by the "fences" placed in the channel of approach, considerable time has been spent in searching for a single formula that would provide for all cases. Of course, such a formula would have to include a velocity term, and could not contain merely the dimensions of the weir and the head.

The possibilities of the formula,

$$Q = 3.33\, L \left(h + a \frac{v^2}{2\,g} \right)^{\frac{3}{2}}$$

in which, v is the mean velocity in the channel of approach, were investigated in spite of the poor showing of the older formulas that follow this type or use it as a basis. It was thought that a suitable formula for a might be found. Consequently, the values of a were computed for all the runs, and plotted against various functions of the heads, heights of weir, ratios of head to height, ratios of head to total depth, etc. Note that this a is not the same as Bazin's α, which latter implies a variable μ (value of m for a weir of infinite height in $Q = m\, L\, h \sqrt{2\,g\,h}$).

There is a general tendency toward an increase in the value of a as the height of the weir increases, as found by earlier experimenters. Apparently, however, unless the distribution of velocities in the channel of approach be rigidly controlled (a matter of difficulty even for a laboratory weir with long approach), a simple law for a through a wide range of heights and heads is hardly to be expected.

Low Weirs with High Heads.—If only the five low weirs, 0.5 ft. to 2.0 ft. in height, of the writers' Series D to O be considered with only the high heads (head as great or greater than the height), the formula,

$$Q = 3.33\, L \left[h + (1.2 + 0.12\, d_0) \frac{v^2}{2\,g} \right]^{\frac{3}{2}} \text{ foot-second units} \ldots\ldots (A)$$

fits closely for the "regular" runs. In only 3 cases out of 50 is the calculated discharge more than 1% from the measured values.

The writers recognized that the influence of the distribution of velocities in the channel of approach to a weir needed experimental study.* Consequently, they made current-meter measurements of velocities a part of their program. It seemed reasonable to suppose that the chief assistance to the static head on a weir crest is that given by the velocity head of that part of the approaching water above the crest level. Therefore, the formula,

$$Q = 3.33\ L \left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

was tested by the new data. In this formula, v_a represents the true mean velocity in the channel of approach above the crest level, or, more strictly, $\frac{v_a^2}{2g}$ is the mean of the velocity heads above crest level, in case the difference between the two values is appreciable. The velocities are to be measured in the same cross-section as that in which the static head is measured.

Fig. 17 shows this formula tested by the new data, including all cases with specially forced velocity distributions by "fences" placed in the channel of approach. The formula fails chiefly, for the "regular" runs, by giving too low discharges for the weirs less than 3 ft. high when the head is greater than the height of weir. For the weirs more than 2.5 ft. high, including Series A, B, and C, all discharges, except 3 out of 126, are computed within 2% and the 3 exceptions are within 3 per cent. This includes all heads and all cases of abnormal velocity distributions for the weirs more than 2.5 ft. high.

For the five low weirs (Series E, N, F, G, and H), with heads higher than the weir, considered in an independent group, it appears that the formula,

$$Q = 3.33\ L \left[h + \left(1 + \frac{\sqrt{d_0}}{4}\right)\frac{v_a^2}{2g}\right]^{\frac{3}{2}} \text{ foot-second units} \ldots\ldots (C)$$

fits closely, but not so well as Equation (*A*).

After numerous trials with other functions of $\frac{v_a^2}{2g}$, it seemed obvious that the energy of that part of the approaching water below the crest level could not be disregarded. It seemed logical to assume that some function of $\frac{v_b^2}{2g}$ (v_b is the mean velocity in the channel of approach below the crest level) could be regarded as assisting the static head, but no suitable formula with a multiplier or other function of $\frac{v_b^2}{2g}$ within the same parenthesis as h and $\frac{v_a^2}{2g}$ was found. Perhaps the best tentative formula found was:

$$Q = 3.33\ L \left[\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} + \left(\frac{v_b^2}{2g} \times h\right)\right] \text{ foot-second units} \ldots\ldots (D)$$

This formula seems to have no exact theory supporting it. However, the additive quantity $\left(\frac{v_b^2}{2g} \times h\right)$ is at least in the nature of a volume.

* Some interesting quotations from hydraulic literature on this topic are given in Appendix C.

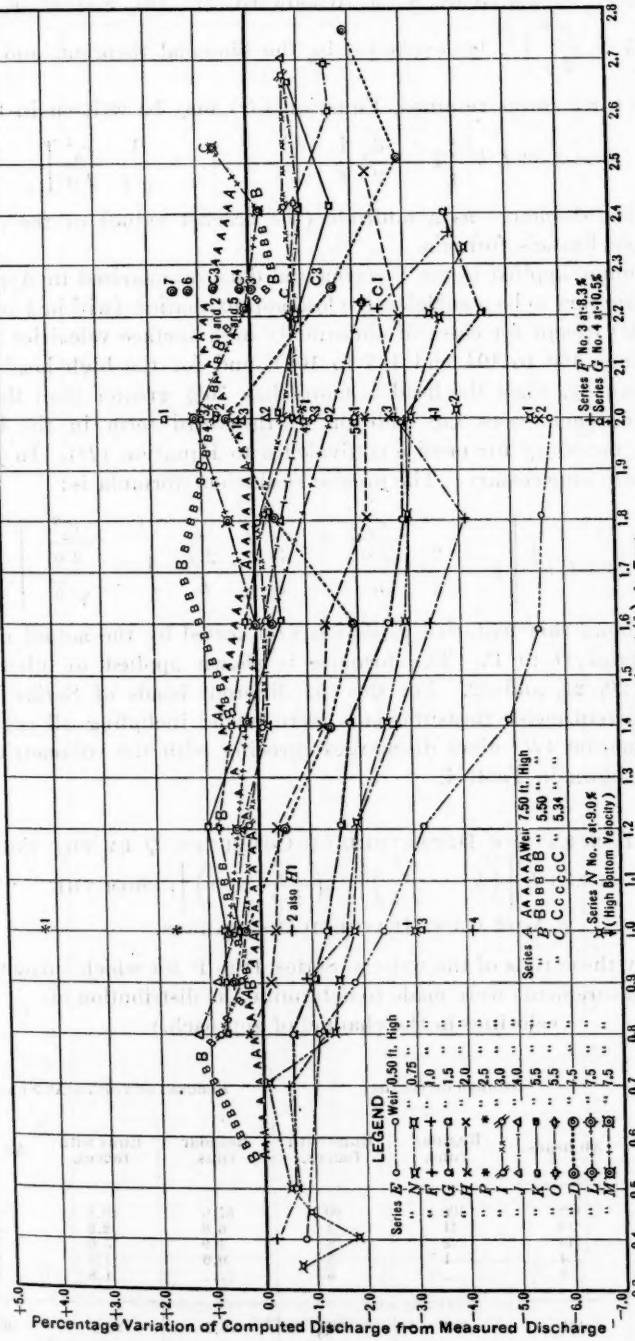
Percentage Variation of Computed Discharge from Measured Discharge
Observed Head (h) in Feet
LEGEND
Series E Weir 0.50 ft. High
Series N " 0.75 " "
Series F " 1.0 " "
Series G " 1.5 " "
Series H " 2.0 " "
Series P " 2.5 " "
Series I " 3.0 " "
Series J " 4.0 " "
Series A Weir 7.50 ft. High
Series B " 5.50 " "
Series C " 5.34 " "
Series K " 5.5 " "
Series O " 5.5 " "
Series D " 7.5 " "
Series L " 7.5 " "
Series M " 7.5 " "
Series N No. 2 at 9.0%
(High Bottom Velocity)
Series F No. 3 at 8.3%
Series G No. 4 at 10.5%
2 also H1

FIG. 17.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF DISCHARGE COMPUTED BY EQUATION (*D*) FROM MEASURED DISCHARGE.

It has been suggested by S. M. Woodward, M. Am. Soc. C. E., that, if the term, $\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}}$, be expanded by the binomial theorem, and only the first two or three terms retained, Equation (*D*) may be written in the form,

$$Q = 3.33\ L\ h^{\frac{3}{2}}\left[1 + \frac{3}{2}\ \frac{1}{h}\ \frac{v_a^2}{2g} + \ldots + \frac{1}{\sqrt{h}}\ \frac{v_b^2}{2g}\right]$$

This gives the discharge as a multiple (the bracket value) of the discharge by the simple Francis formula.

This formula, applied to the experimental data summarized in Appendix B (Table 55) appears to be a sufficiently close approximation (within ¼ of 1%) to Equation (*D*) except for cases of abnormally high surface velocities (such as Series C, Runs 100 to 104 and 105 to 109), and for the high heads on the low weirs, namely, when the head is more than 50% greater than the height of weir. For such cases the addition of the third term in the binomial expansion is necessary for precise equivalence to Equation (*D*). In practice, more terms are unnecessary. The precise equivalent formula is:

$$Q = 3.33\ L\ h^{\frac{3}{2}}\left[1 + \frac{3}{2}\ \frac{\frac{v_a^2}{2g}}{h} + \frac{3}{8}\left(\frac{\frac{v_a^2}{2g}}{h}\right)^2 + \frac{\frac{v_b^2}{2g}}{\sqrt{h}}\right] \ldots\ldots (E)$$

Fig. 18 shows this tentative Equation (*D*) tested by the actual measurements in Series D to P. The formula is shown applied to other Series in Figs. 19, 20, 21, and 22. For the 161 different heads of Series D to P for which current-meter measurements were made, including all cases with "fences", Equation (*D*) gives discharges agreeing with the volumetric measurements as shown in Table 4.

TABLE 4.—PERCENTAGE DEPARTURE OF COMPUTED Q BY THE FORMULA, $Q = 3.33\ L\left[\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} + \left(\frac{v_b^2}{2g} \times h\right)\right]$, FROM THE ACTUALLY MEASURED DISCHARGES.

(Includes only those runs of the writers' Series D to P for which current-meter measurements were made to determine the distribution of velocities in the channel of approach.)

Variation.	Number of Heads			Percentage of Total Number.		
	All heads.	Regular runs.	Runs with fences.	Regular runs.	Runs with fences.	All heads.
1% or less	128	108	20	67.0	12.5	79.5
1–1.5%	15	11	4	6.8	2.5	9.3
1.5–2%	11	3	8	1.9	5.0	6.9
2.0–3%	4	1	3	0.6	1.9	2.5
3.0–5.3%	3	....	3	....	1.8	1.8
Total	161	123	38	76.3	23.7	100.0

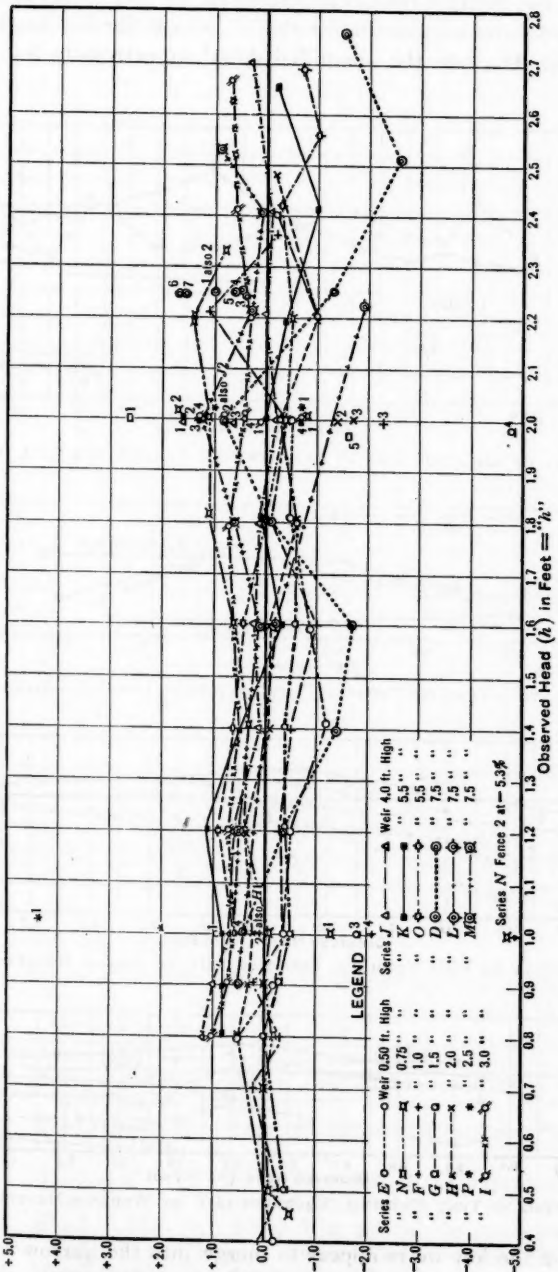
Percentage Variation of Computed Discharge from Measured Discharge
Observed Head (h) in Feet = "h"
LEGEND
Series E Weir 0.50 ft. High
Series J Weir 4.0 ft. High
Series N Fence 2 at — 5.35

FIG. 18.—SHARP-CRESTED WEIRS. PERCENTAGE OF VARIATION OF DISCHARGE COMPUTED BY EQUATION (*D*) FROM MEASURED DISCHARGE.

Low Heads on Sharp-Crested Weirs.—Fig. 10 shows in detail the percentage departures and coefficients for Series D to O for low heads (less than 0.5 ft.), and Fig. 11 gives the magnified detail at extremely low heads (less than 0.2 ft.).

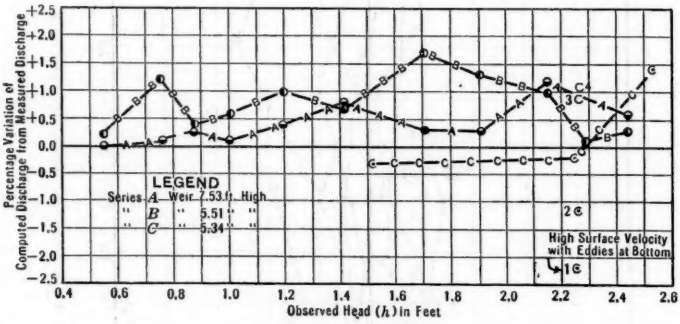

FIG. 19.—TEST OF EQUATION (*D*) BY SCHODER AND TURNER, SERIES A, B, AND C.

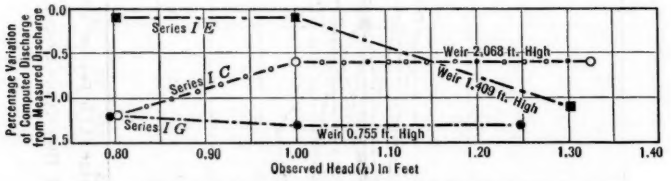

FIG. 20.—TEST OF EQUATION (*D*) BY MEYER AND SEE'S DATA.

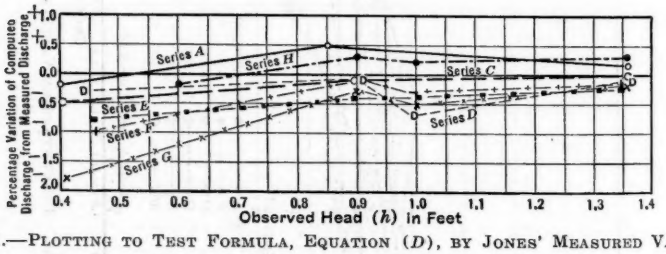

FIG. 21.—PLOTTING TO TEST FORMULA, EQUATION (*D*), BY JONES' MEASURED VALUES.

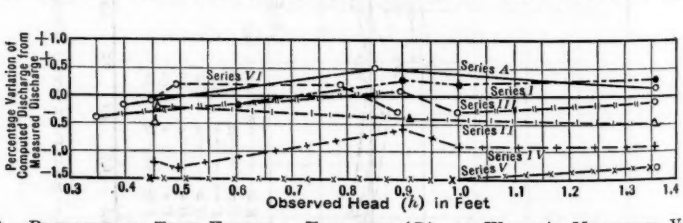

FIG. 22.—PLOTTING TO TEST FORMULA, EQUATION (*D*), BY WEBER'S MEASURED VALUES.

The lines for the low weirs appear to merge into the narrow zone common to all weirs at a head equal to about one-fifth the height of the weir.

In studying Figs. 10 and 11, the considerable effect of small errors in measurement is to be remembered; for example, an error of 0.0005 ft. in

measuring a head of 0.05 ft. causes an error of 1.5% in the computed discharge. If the width of the zone of lines for the various weirs covers 2% at 0.1 ft. head, or if the extreme edges are only 1% from the average line, it indicates that the low heads have been correctly measured within two-thirds of 1%, or 0.0007 ft. (assuming all the error to be in the head).

The writers' data on low heads in Series C (crest width = $\frac{3}{8}$ in., edge dulled by service) are shown on Fig. 23. The peculiar clinging phenomena at the extremely low heads are illustrated. The details of the conditions at the crest in the various tests of Fig. 23 are, as follows:

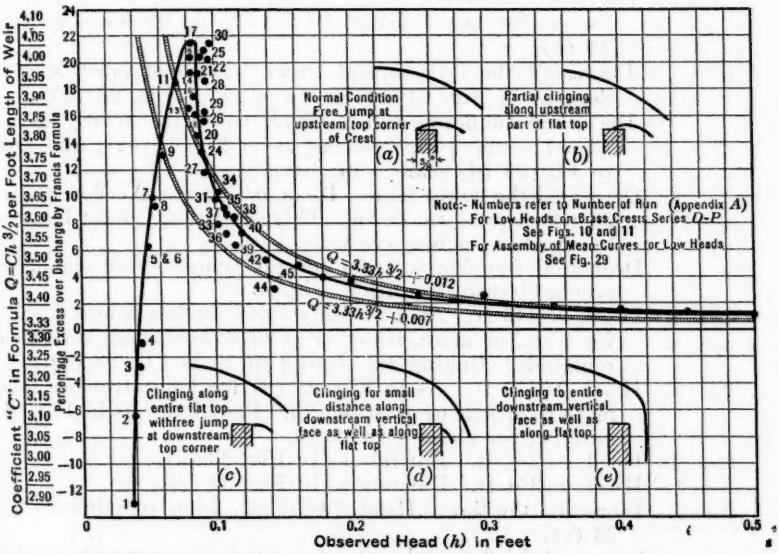
Coefficient "C" in Formula Q=Ch 3/2 per Foot Length of Weir
Percentage Excess over Discharge by Francis Formula
Observed Head (h) in Feet
Normal Condition Free Jump at upstream top corner of Crest
(a)
Partial clinging along upstream part of flat top
(b)
Note:- Numbers refer to Number of Run (Appendix A)
For Low Heads on Brass Crests Series D-P
See Figs. 10 and 11
For Assembly of Mean Curves for Low Heads
See Fig. 29
Q = 3.33h 3/2 + 0.012
Q = 3.33h 3/2 + 0.007
Clinging along entire flat top withfree jump at downstream top corner
(c)
Clinging for small distance along downstream vertical face as well as along flat top
(d)
Clinging to entire downstream vertical face as well as along flat top
(e)

FIG. 23.—LOW-HEAD WEIR EXPERIMENTS, PART OF SERIES C, SHOWING DEPARTURES FROM DISCHARGE LAWS APPLICABLE TO HIGHER HEADS AND UNCERTAIN DISCHARGE CONDITIONS IN CRITICAL REGIONS OF VERY LOW HEADS.

Tests of March 16, 1914:

No. 24. Head, 0.0912 ft. Flow conditions as in Fig. 23 (*c*) for three-fourths of crest length and as in Fig. 23 (*b*) for remainder in local stretches $\frac{1}{2}$ in. long.

No. 20. Head, 0.0879 ft. A little more clinging than in Test No. 24.

No. 13. Head, 0.0813 ft. As in Fig. 23 (*c*), except for about ten $\frac{1}{2}$-in. stretches as in Fig. 23 (*b*).

No. 19. Head, 0.0878 ft. Less clinging than in Test No. 13, but more than in Test No. 20.

No. 26. Head, 0.0927 ft. As in Test No. 19. Considerable more clinging than in Tests Nos. 24 or 20.

No. 33. Head, 0.1020 ft. As in Fig. 23 (*b*) in stretches totaling one-half of crest length. Remainder as in Fig. 23 (*c*).

No. 36. Head, 0.1083 ft. As in Fig. 23 (*b*) more than half of length. As in Fig. 23 (*c*) in short stretches $\frac{1}{4}$ in. long.

No. 39. Head, 0.1156 ft. As in Fig. 23 (*b*), except an occasional patch as in Fig. 23 (*c*).

Tests of March 17, 1914:

No. 28. Head, 0.0936 ft. Flow conditions as in Fig. 23 (*c*).

Increased flow a trifle. Head held 0.0035 ft. higher than in Test No. 28 as in Fig. 23 (*c*) for a few moments, but then broke into condition, Fig. 23 (*b*), in short stretches totaling more than one-fourth of crest length. Head then settled at 0.1037 ft., or 0.0101 ft. higher than in Test No. 28.

Decreased flow a trifle. Head, 0.099 ft. No change apparent.

Decreased flow a trifle. Head, 0.0955 ft. Flow as in Fig. 23 (*c*) for a little more than half of length with remainder as in Fig. 23 (*b*).

No. 27. Decreased the flow a trifle. Head, 0.0934 ft. As in Fig. 23 (*c*) except for about thirty ¼-in. to ½-in. stretches as in Fig. 23 (*b*).

Decreased the flow several times. More of crest length has flow, as in Fig. 23 (*c*), and less, as in Fig. 23 (*b*).

Decreased the flow a trifle. Head, 0.0833 ft. Mostly as in Fig. 23 (*c*); some ¼-in. stretches as in Fig. 23 (*b*); and some as in Fig. 23 (*d*) scant ¼ in. down.

Decreased the flow a trifle. Head, 0.0793 ft. All flow as in Fig. 23 (*d*) scant ¼ in. down. This condition cannot be changed by brushing across crest.

Decreased the flow a trifle several times. Flow changing in patches toward condition, Fig. 23 (*e*).

No. 4. Head, 0.0445 ft. About twenty-five stretches where flow clings for ¾ in., or more, below crest to down-stream vertical face, remainder clinging for about ¼ in. as in Fig. 23 (*d*).

No. 3. Head, 0.0419 ft. As in Fig. 23 (*e*) for long stretches totaling more than half of crest length. Remainder as in Fig. 23 (*d*) clinging for about ¼ in. down.

No. 2. Head, 0.0384 ft. Entirely as in Fig. 23 (*e*).

No. 1. Head, 0.0364 ft. Entirely as in Fig. 23 (*e*).

No. 7. Increased the flow. Head, 0.0537 ft. Flow remains as in Fig. 23 (*e*).

Increased the flow and with head, 0.059, flow jumps to condition, Fig. 23 (*d*), clinging to vertical down-stream face for second ¼ in.

Increased the flow several times, but no further change.

No. 18. Head, 0.0853 ft. Flow as in Test No. 7.

Increased the flow several times and in patches it changes to condition, Fig. 23 (*c*), with head of 0.099 ft. Then, with head of 0.102 ft., flow changes to condition, Fig. 23 (*c*), for one-third of crest length and as in Fig. 23 (*b*) for remainder in forty-eight patches, ¼ in. to 1¾ in. long, and head settles at 0.1053 ft.

Increased the flow. Head, 0.118 ft. one-fifth of length as in Fig. 23 (*c*), remainder as in Fig. 23 (*b*).

Increased the flow. Head, 0.132 ft. As in Fig. 23 (*b*), except for some short patches as in Fig. 23 (*c*).

No. 44. Increased the flow. Head, 0.1438 ft. Flow jumps free as in Fig. 23 (*a*), except for ⅛ in. to ⅜ in., where it clings to flat top as in Fig. 23 (*b*).

Increased the flow. Head, 0.168 ft. As before, but clings to flat top for ⅛ to ⅜ in. only in spots.

Increased the flow. Head, 0.196 ft. Conditions as before (for 0.168-ft. head).

Similar data on low heads, down to about 0.04 ft., from Meyer and See's experiments, are shown on Figs. 24 and 25. On these diagrams, the points marked *A* represent measurements from a series of progressively increased low

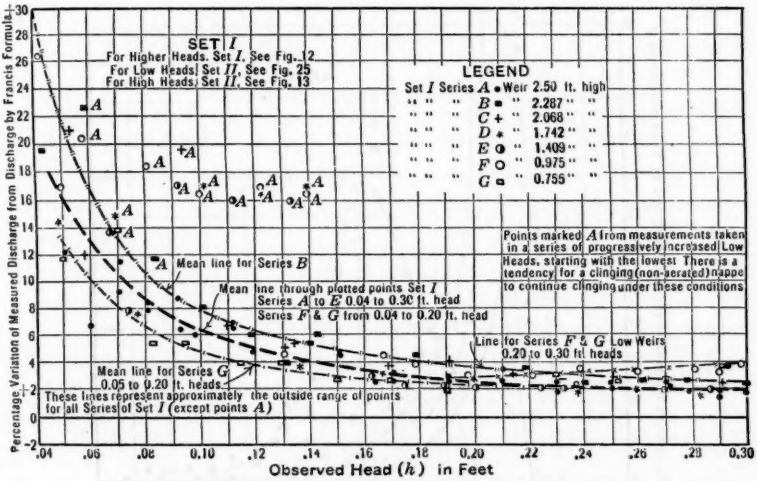
SET I
For Higher Heads, Set I, See Fig. 12
For Low Heads, Set II, See Fig. 25
For High Heads, Set II, See Fig. 13
LEGEND
Set I Series A Weir 2.50 ft. high
" " " B " 2.287 " "
" " " C " 2.068 " "
" " " D " 1.742 " "
" " " E " 1.409 " "
" " " F " 0.975 " "
" " " G " 0.755 " "
Points marked A from measurements taken in a series of progressively increased Low Heads, starting with the lowest. There is a tendency for a clinging (non-aerated) nappe to continue clinging under these conditions
Mean line for Series B
Mean line through plotted points Set I Series A to E 0.04 to 0.30 ft. head Series F & G from 0.04 to 0.20 ft. head
Line for Series F & G Low Weirs 0.20 to 0.30 ft heads
Mean line for Series G 0.05 to 0.20 ft. heads
These lines represent approximately the outside range of points for all Series of Set I (except points A)
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Observed Head (h) in Feet

FIG. 24.—LOW-HEAD EXPERIMENTS BY MEYER AND SEE, SET I, SERIES A TO G.

heads starting with the lowest. There is a tendency for a clinging (non-aerated) nappe to continue clinging under these conditions. These points, *A*, were not considered in drawing the curves of Figs. 24 and 25.

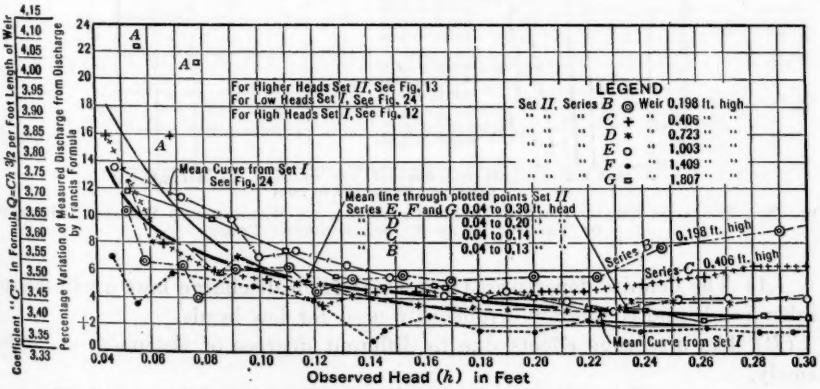
For Higher Heads Set II, See Fig. 13
For Low Heads Set I, See Fig. 24
For High Heads Set I, See Fig. 12
LEGEND
Set II, Series B Weir 0.198 ft. high
" " " C " 0.406 " "
" " " D " 0.723 " "
" " " E " 1.003 " "
" " " F " 1.409 " "
" " " G " 1.807 " "
Mean Curve from Set I See Fig. 24
Mean line through plotted points Set II
Series E, F and G 0.04 to 0.30 ft. head
D 0.04 to 0.20 " "
C 0.04 to 0.14 " "
B 0.04 to 0.13 " "
Series B
0.198 ft. high
Series C
0.406 ft. high
Mean Curve from Set I
Coefficient "C" in Formula Q=CbH 3/2 per Foot Length of Weir
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Observed Head (h) in Feet

FIG. 25.—LOW-HEAD EXPERIMENTS BY MEYER AND SEE, SET II, SERIES B TO G.

Data from Walker and Weidner's experiments at low heads on several widths of crest piece are shown on Fig. 26. Fig. 27 shows Fteley and Stearns' observed data* for low heads and the basis for the averaging curves used in other plottings. Fig. 28 is a partial assembly of low-head data, showing the observed values of Bazin in comparison with those of others. The crests ranged from known definitely sharp conditions through known dulled corners and uncertain cases to known definitely rounded corners.

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), pp. 25–28.

Fig. 29 is a general assembly of curves representing all the low-head data on crests with sharp, dull, uncertain, and slightly rounded, top, up-stream, corners. It shows:

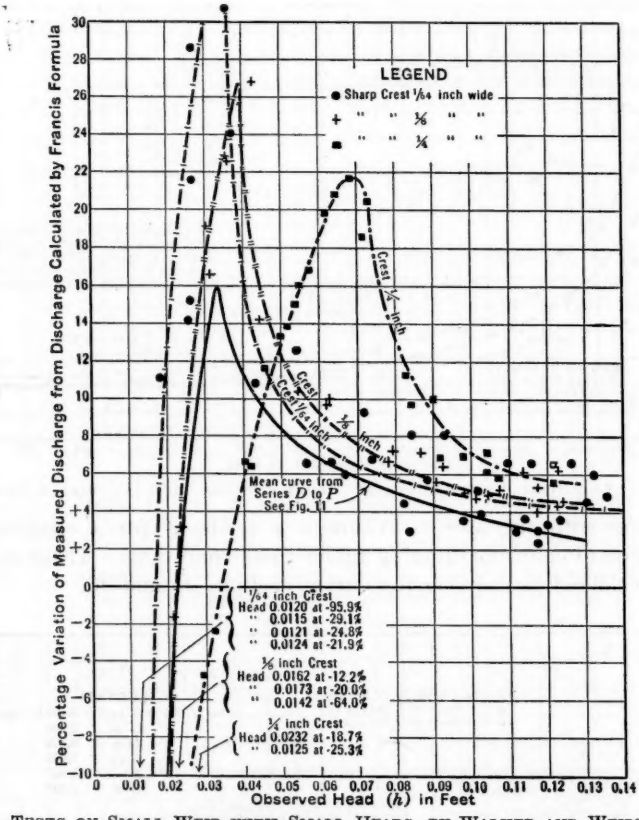
LEGEND
Sharp Crest 1/64 inch wide
" " 1/16 " "
" " 1/4 " "
Percentage Variation of Measured Discharge from Discharge Calculated by Francis Formula
Observed Head (h) in Feet
Crest 1/2 Inch
Crest 1/16 Inch
Crest 1/4 Inch
Mean curve from Series D to P See Fig. 11
1/64 inch Crest
Head 0.0120 at -95.9%
" 0.0115 at -29.1%
" 0.0121 at -24.8%
" 0.0124 at -21.9%
1/16 inch Crest
Head 0.0162 at -12.2%
" 0.0173 at -20.0%
" 0.0142 at -64.0%
1/4 inch Crest
Head 0.0232 at -18.7%
" 0.0125 at -25.3%

FIG. 26.—TESTS ON SMALL WEIR WITH SMALL HEADS, BY WALKER AND WEIDNER. EFFECTS OF VARIATIONS IN WIDTH OF CREST PIECE.

(*A*) The definite departures (increases) from discharge laws applicable to higher heads and commonly erroneously used for low heads.

(*B*) The discharge effects due to different degrees of sharpness of crests, namely,

(1) Crests generally accepted as strictly sharp and so described by experimenters:
   (*a*) Francis.
   (*b*) Fteley and Stearns.

(2) Crests generally accepted as strictly sharp, but with experimenter's description incomplete and indefinite:
   (*a*) Bazin.

(3) Crests known to writers to be strictly sharp:
   (*a*) Schoder and Turner (Series D to P).
   (*b*) Schoder and Turner, Series B.

(*c*) Meyer and See, Series I and II.
(*d*) J. O. Jones.
(*e*) Walker and Weidner.

(4) Crests known to writers to be dulled by service conditions:

(*a*) Schoder and Turner, Series A.
(*b*) Schoder and Turner, Series C.
(*c*) Dawson, Series I.

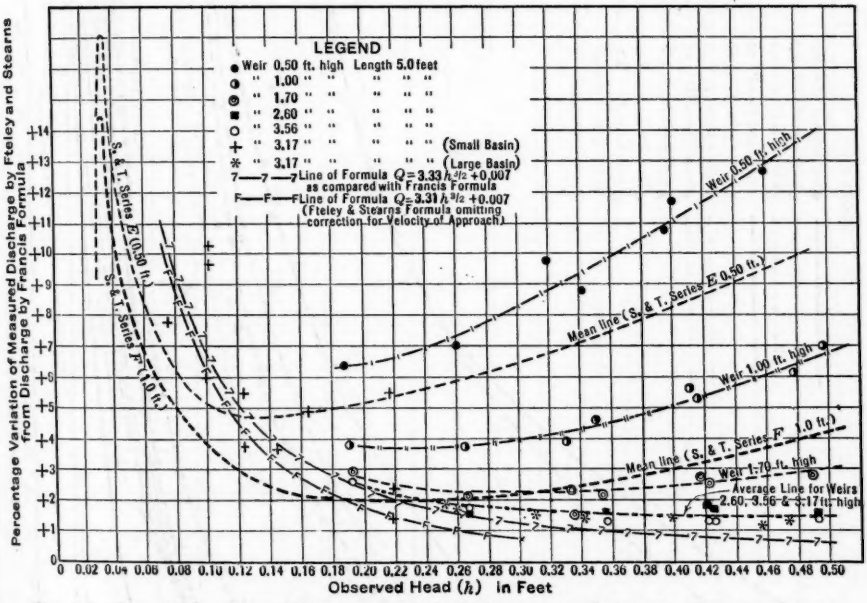
LEGEND
Weir 0.50 ft. high Length 5.0 feet
" 1.00 " " " " "
" 1.70 " " " " "
" 2.60 " " " " "
" 3.56 " " " " "
" 3.17 " " " " " (Small Basin)
" 3.17 " " " " " (Large Basin)
7—7—7 Line of Formula $Q=3.33\,h^{3/2}+0.007$ as compared with Francis Formula
F—F—F Line of Formula $Q=3.31\,h^{3/2}+0.007$ (Fteley & Stearns Formula omitting correction for Velocity of Approach)
S. & T. Series E (0.50 ft.)
S. & T. Series F (1.0 ft.)
Weir 0.50 ft. high
Mean line (S. & T. Series E 0.50 ft.)
Weir 1.00 ft. high
Mean line (S. & T. Series F 1.0 ft.)
Weir 1.70 ft. high
Average Line for Weirs 2.60, 3.56 & 3.17 ft. high
Percentage Variation of Measured Discharge by Fteley and Stearns from Discharge by Francis Formula
+14 +13 +12 +11 +10 +9 +8 +7 +6 +5 +4 +3 +2 +1 0
0 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0.16 0.18 0.20 0.22 0.24 0.26 0.28 0.30 0.32 0.34 0.36 0.38 0.40 0.42 0.44 0.46 0.48 0.50
Observed Head (h) in Feet

FIG. 27.—LOW-HEAD WEIR EXPERIMENTS, BY FTELEY AND STEARNS. COMPARISON OF RESULTS WITH SCHODER AND TURNER ON BASIS OF FRANCIS FORMULA.

(5) Crests known to writers to be definitely and purposely rounded:

(*a*) J. O. Jones, $\frac{1}{64}$ in., round.
(*b*) J. O. Jones, $\frac{1}{24}$ in., round.
(*c*) Dawson, $\frac{1}{8}$ in., round.
(*d*) Dawson, $\frac{1}{4}$ in., round.
(*e*) Fteley and Stearns, $\frac{1}{4}$ in., round.

(*C*) The discharge effects (at very low heads) due to different top widths of crest piece, where clinging effects are involved:

(*a*) Walker and Weidner, $\frac{1}{64}$ in. wide.
(*b*) Walker and Weidner, $\frac{1}{8}$ in. wide.
(*c*) Walker and Weidner, $\frac{1}{4}$ in. wide.
(*d*) Schoder and Turner, Series D to P, $\frac{3}{32}$ in. wide.
(*e*) Schoder and Turner, Series B, $\frac{3}{8}$ in. wide.
(*f*) Schoder and Turner, Series C, $\frac{3}{8}$ in. wide.

All these new data show a general agreement with the findings of earlier experimenters that there is an increasing percentage increase in discharge as the head decreases below about 0.4 ft. for sharp-crested weirs having heights of more than 2 ft. The law of increase in discharge according to Fteley and

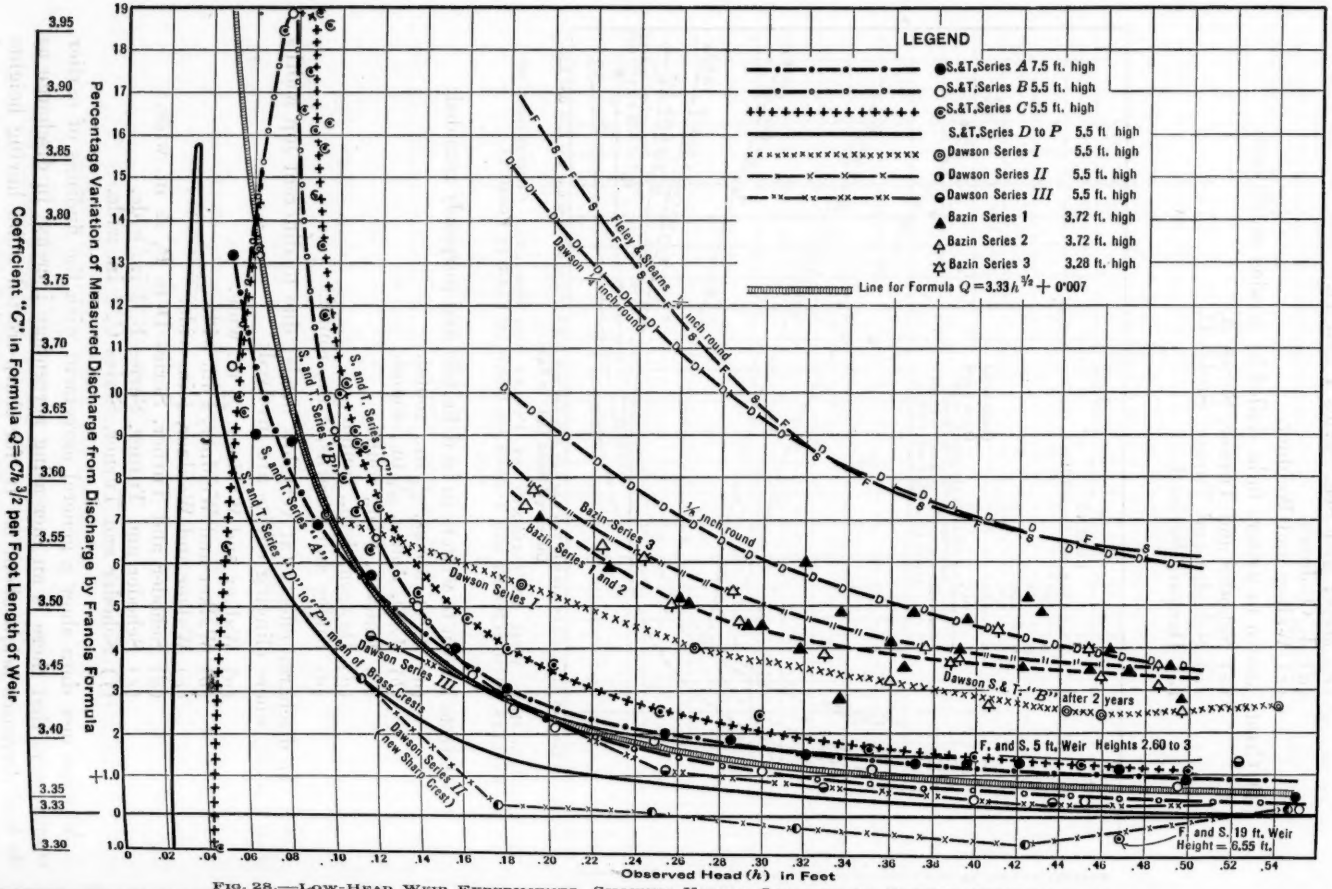
LEGEND
S.&T. Series A 7.5 ft. high
S.&T. Series B 5.5 ft. high
S.&T. Series C 5.5 ft. high
S.&T. Series D to P 5.5 ft high
Dawson Series I 5.5 ft. high
Dawson Series II 5.5 ft. high
Dawson Series III 5.5 ft. high
Bazin Series 1 3.72 ft. high
Bazin Series 2 3.72 ft. high
Bazin Series 3 3.28 ft. high
Line for Formula Q = 3.33 h^3/2 + 0·007
Coefficient "C" in Formula Q = Ch^3/2 per Foot Length of Weir
Percentage Variation of Measured Discharge from Francis Formula
Observed Head (h) in Feet
Fteley & Stearns ¼ inch round
Dawson ¼ inch round
¼ inch round
Bazin Series 3
Bazin Series 1 and 2
Dawson Series I
Dawson Series III
S. and T. Series "A"
S. and T. Series "B"
S. and T. Series "C"
S. and T. Series "D" to "P" mean of Brass Crests
Dawson Series II (new Sharp Crest)
Dawson S. & T. "B" after 2 years
F. and S. 5 ft. Weir Heights 2.60 to 3
F. and S. 19 ft. Weir Height = 6.55 ft.

FIG. 28.—LOW-HEAD WEIR EXPERIMENTS, SHOWING VALUES OBTAINED BY VARIOUS EXPERIMENTERS.

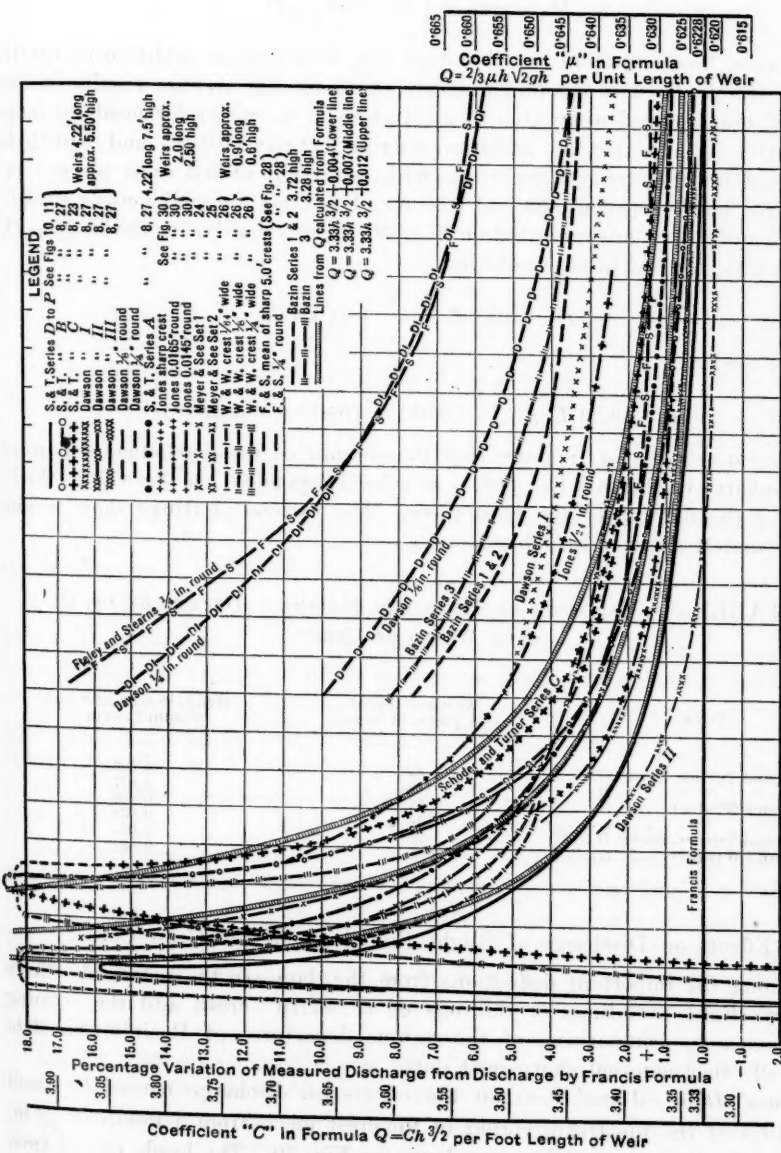
LEGEND
Fteley and Stearns ¾ in. round
Dawson ¾ in. round
Schoder and Turner Series C
Dawson Series I
Jones ½ in. round
Basin Series 1 & 2
Basin Series 3
Dawson Series II
Francis Formula
Coefficient "μ" in Formula Q = 2/3 μh √2gh per Unit Length of Weir
Observed Head (h) in Feet
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Coefficient "C" in Formula Q = Ch 3/2 per Foot Length of Weir

FIG. 29.—LOW-HEAD WEIR EXPERIMENTS. ASSEMBLY OF MEAN CURVES.

FIG. 28.—LOW-HEAD WEIR EXPERIMENTS, SHOWING VALUES OBTAINED BY VARIOUS EXPERIMENTERS.

Stearns is expressed by adding $0.007L$ (foot-second units) to their basic expression,

$$Q = 3.31\ L\left(h + 1.5\frac{v^2}{2g}\right)^{\frac{3}{2}}$$

It appears, however, from the abundant new data that an addition of $0.004L$ to the simple Francis formula properly expresses the average results on the writers' brass-crested weirs (top of crest piece $\frac{3}{32}$ in. wide) for heads of more than 0.08 ft. and that an addition ranging between $0.010L$ and $0.012L$ is indicated for the writers' Series C (with somewhat dulled crest piece $\frac{3}{8}$ in. wide) for heads between 0.09 and 0.30 ft. The lines for additions of $0.004L$, $0.007L$, and $0.012L$ are indicated on Fig. 29. Consequently, for Series D to O, the formula for low heads would be,

$$Q = 3.33\ L \times h^{\frac{3}{2}} + 0.004\ L$$

and for Series C,

$$Q = 3.33\ L \times h^{\frac{3}{2}} + 0.011\ L \text{ (foot-second units)}$$

The interesting fact appears that the summit of the percentage departure (or discharge coefficient) line occurs at a head dependent on the width (thickness) of the flat top of the crest piece. The various plottings show results approximately as given in Table 5.

TABLE 5.—Variation of Head for Maximum Departure of Q, with Width of Crest.

Tests.	Width of top of crest piece, in inches.	Head, in feet, at which summit occurs.
Schoder and Turner, Series C.....	$\frac{3}{8}$	0.085
" " " " B.....	$1\frac{1}{32}$	0.077
Walker and Weidner.....	$\frac{1}{4}$	0.068
" " "	$\frac{1}{8}$	0.039
Schoder and Turner, Series D to P	$\frac{5}{32}$	0.035
Walker and Weidner.....	$\frac{1}{64}$	0.032

Effects on Discharge of Slight Roundings of the Crest Corner

Among the important deductions from the data are the noticeable effects of very slight roundings or dullings of a "sharp" crest, and the seeming significance by comparison, of the unique departures of Bazin's weir data from all other nominal sharp-crest data.

Jones' Data.—Jones' data on the effects on discharge caused by small roundings of the up-stream corner of the crest piece, from a radius of $\frac{1}{24}$ in. down to sharp-crest condition, are shown on Fig. 30. The heads ranged from 0.18 ft. to 1.36 ft. on a weir 2.5 ft. high and 2 ft. long. The percentage increase in discharge, due to rounding, over sharp-crest values is seen to decrease with the head. It amounts to 1.6% for the $\frac{1}{24}$-in. radius rounding at 0.5-ft. head, and to 0.7% at 1.0-ft. head. A rounding with $\frac{1}{60}$-in. radius

causes an increase of 0.85% at 0.5-ft. head and 0.45% at 1-ft. head. Still smaller roundings with less than $\frac{1}{100}$-in. radius, cause definitely measurable effects. For the range of his experiments, Jones found the formula,

$$\text{Percentage increase in } Q = 22.8 \frac{R}{h^{3/4}}$$

to hold closely. R is the radius, in inches, of the quadrant rounding of the up-stream top corner, and h is the head, in feet, on the weir.

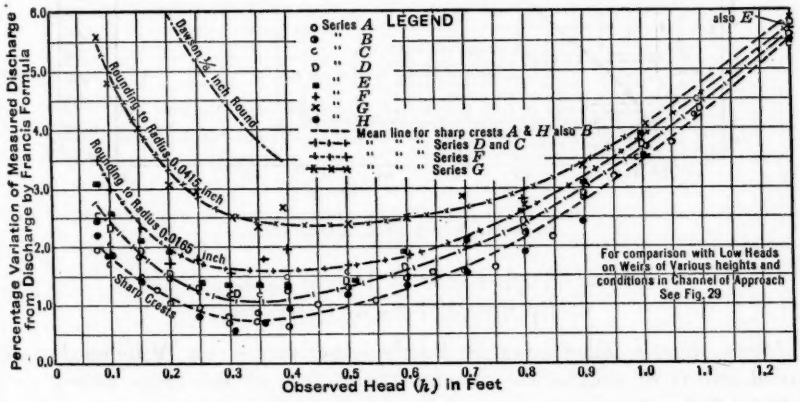
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Observed Head (h) in Feet
LEGEND
Series A
" B
" C
" D
" E
" F
" G
" H
Mean line for sharp crests A & H also B
" " " Series D and C
" " " Series F
" " " Series G
Dawson 1/8 inch Round
Rounding to Radius 0.0415 inch
Rounding to Radius 0.0165 inch
Sharp Crests
also E
For comparison with Low Heads on Weirs of Various heights and conditions in Channel of Approach See Fig. 29

FIG. 30.—EFFECTS ON DISCHARGE DUE TO SMALL ROUNDINGS OF EDGE OF CREST, JONES' DATA.

Fig. 28 represents a study in which the data of Bazin's volumetric measurements* of his Series I, II, and III are shown in comparison with data of several weirs the crests of which ranged from definite roundings with $\frac{1}{4}$-in. radius down through Jones' smaller roundings and still further down through appreciable dullings to strictly sharp conditions. Only low heads are considered, in order to eliminate as far as possible the effects of probable variations, but of unknown degree, in distributions of velocities in the channels of approach.

Bazin's Results.—These results lie with remarkable consistency between Dawson's $\frac{1}{8}$-in. radius rounding and Jones' $\frac{1}{24}$-in. rounding and are far separated from the sharp-crest data. His discharges are distinctly higher than those of crests known to be dulled and perceptibly rounded; namely, about 1% higher than Dawson's Series I, and 3 to 4% higher than the writers' Series A and C, which themselves have greater discharges than the sharp crests.

Fig. 31 represents a comparison, all at 0.5-ft. head, of the discharges over crests rounded to various radii for weirs more than 3 ft. high, and also shows the discharges for crests with edges known to be dulled and for sharp crests. Again, Bazin's data appear in a unique relation to the other data.

* Bazin described his crest piece as an iron plate "bien dressée," 0.007 m. (0.276 in.) thick. The "bien dressée" is translated by Marichal and Trautwine in *Proceedings*, Engineers Club of Philadelphia, January, 1890, as "carefully straightened." The drawing, Fig. 9, Plate 2, in *Annales des Ponts et Chaussées*, October, 1888, shows the plate to be about 7.4 in. wide, with top edge square and flat. The title reads, "Section of the crest of the weir of sharp-edged type" ("type à arête vive").

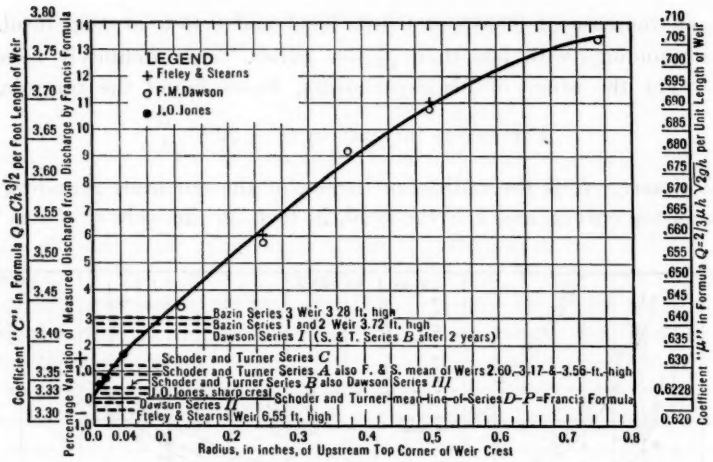
LEGEND
+ Fteley & Stearns
o F.M.Dawson
• J.O.Jones
Bazin Series 3 Weir 3.28 ft. high
Bazin Series 1 and 2 Weir 3.72 ft. high
Dawson Series I (S. & T. Series B after 2 years)
Schoder and Turner Series C
Schoder and Turner Series A also F. & S. mean of Weirs 2.60,-3.17-&-3.56-ft.-high
Schoder and Turner Series B also Dawson Series III
J.O.Jones, sharp crest
Schoder and Turner-mean-line-of-Series D-P=Francis Formula
Dawson Series II
Fteley & Stearns Weir 6.55 ft. high
Coefficient "C" in Formula $Q=Ch^{3/2}$ per Foot Length of Weir
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Coefficient "μ" in Formula $Q=2/3\mu k\sqrt{2g}h$ per Unit Length of Weir
Radius, in inches, of Upstream Top Corner of Weir Crest

FIG. 31.—COMPARISON OF ROUND-CRESTED WEIRS WITH SHARP (OR APPROXIMATELY SHARP) CRESTED WEIRS AT 0.50-FT. HEAD.

Effects on Discharge of Degree of Roughness of Up-Stream Face of Crest Piece

James Barr's Experiments.—Barr's experiments on V-notched weirs* showed effects of roughening the up-stream face of the brass weir plate as given in Table 6.

TABLE 6.—RELATION OF DISCHARGE TO ROUGHNESS OF UP-STREAM FACE OF WEIR.

Condition of up-stream surface of notch.	PERCENTAGE INCREASE IN DISCHARGE OVER VALUE FOR PLAIN SURFACE.	
	Head = 3 in.	Head = 4 in.
Plain surface	0.	0.
One coat varnish (shellac)	0.39	0.26
Coated with medium emery	1.8	1.2
Coated with coarse emery	2.4	1.7

Weber's Data.—These data are represented on Fig. 32. The percentage increases in discharge, due to roughening the up-stream face, do not seem to decrease as rapidly with the increase of the head as for the slight roundings of the crest edge. Thus, Series V at 0.5-ft. head shows 1.85% increase over smooth-plate conditions, and even at 1.35-ft. head the increase is still 1.2 per cent.

The part of the plate near the crest appears to have more influence than that farther removed, although the addition of 8 in. of rough surface below the top 4 in. caused a further increase of discharge averaging fully 0.5% for heads between 0.3 ft. and 1.35 ft.

* *Engineering* (London), April 8 and 15, 1910.

Bazin's Discharges Too High for Sharp-Crest and Smooth-Plate Condition

Again returning to Bazin's data (see Table 7), it appears that a combination of slight rounding at the edge and rusting of the up-stream face could have produced just such a percentage effect as Bazin's measurements show when compared with other sharp crests. In fact, Dawson's Series I as compared with the writers' Series B, using the same weir and crest piece, shows nearly the same increase of discharge as Bazin's departure from all the other known sharp crests.

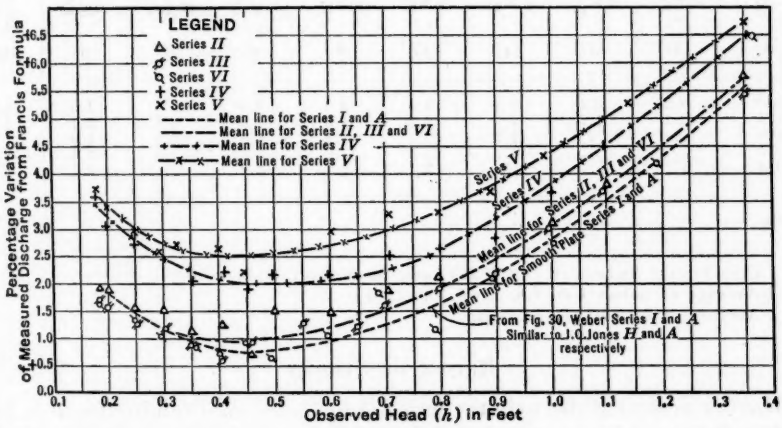
LEGEND
Series II
Series III
Series VI
Series IV
Series V
Mean line for Series I and A
Mean line for Series II, III and VI
Mean line for Series IV
Mean line for Series V
Percentage Variation of Measured Discharge from Francis Formula
Observed Head (h) in Feet
Series V
Series IV
Mean line for Series II, III and VI
Mean line for Smooth Plate Series I and A
From Fig. 30, Weber Series I and A
Similar to J.C. Jones H and A
respectively

FIG. 32.—RELATION OF DISCHARGE TO ROUGHNESS OF UP-STREAM FACE OF CREST PIECE, WEBER'S DATA.

The writers see no reason for questioning Bazin's measurements of heads and discharges in the ratings of his "standard weir" by Series I, II, and III. The fact that he did not divert an already established flow into his measuring basin, but started with a static head, seems to be amply offset by the long duration of his measurements, from 10 to 20 min. Consequently, there is no implied criticism of the great mass of subsequent measurements made by Bazin, using his "standard weir" as a reference except as this may have deteriorated. The only point suggested is that his data on Series I to X do not appear to belong in the sharp-crest, smooth-plate category.

Rehbock* (and Hansen† before him), has supplied experimental evidence showing that Bazin's discharges are too high as compared to their values for sharp-crested weirs. Bazin himself tabulated‡ the excesses of his values over those of Fteley and Stearns for both the latter's 1877 and 1879 experiments.

It should be mentioned that Gardner S. Williams, M. Am. Soc. C. E., reached the conclusion long ago§ that the data of Francis and of Fteley and Stearns were more reliable than those of Bazin.

* Handbuch der Ingenieur Wissenschaften, III Teil, Wasserbau, Band 2 (Stauwerke), Vierte Auflage, 1912, p. 59.

† *Zeitschrift* des Vereins deutscher Ingenieure, Band 36, p. 1057.

‡ *Annales des Ponts et Chaussées*, October, 1888; also, *Proceedings*, Engineers' Club of Philadelphia, January, 1890, p. 279.

§ *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 321.

TABLE 7.—BAZIN'S PERCENTAGE EXCESS DISCHARGE, BY HIS COEFFICIENTS, OVER THE EXPERIMENTAL DATA (OLD AND NEW) BY VARIOUS OBSERVERS.

(In the Headings, B, 3.72 Means Bazin's Weir, 3.72 ft. High; F & S = Fteley and Stearns; D = Dawson; M & S = Meyer and See; S & T = Schoder and Turner. All Nominally Sharp Crests.)

Observed head, in feet. (1)	B, 3.72 over combination.* Percentage. (2)	B, 3.72 over F & S, 3.17. Percentage. (3)	B, 2.46 over F & S, 2.60. Percentage. (4)	B, 2.46 over M & S, 2.50. Percentage. (5)	B, 1.64 over F & S, 1.70. Percentage. (6)	B, 1.64 over M & S, 1.60.† Percentage. (7)	B, 1.16 over Martin, 1.20. Percentage. (8)	B, 0.79 over M & S, 0.74.‡ Percentage. (9)	Average excess. Percentage. (10)
0.3	4.2	3.1	3.3	3.2	3.0	3.3	3.5	1.8	3.2
0.5	3.1	1.7	2.1	2.1	1.9	1.8	2.6	2.1	2.2
0.8	...	0	1.2	2.4	1.1	2.6	2.3	1.9	1.9
1.0	...	...	...	2.7	...	2.5	2.5	3.3	2.7
1.2	...	...	...	2.6	...	2.7	...	3.5	2.9
Average..	...	...	...	...	...	...	...	...	2.6

* Combination includes F & S, S & T, and D, the average being 5.85.
† Average of Series I-D, I-E, II-F, and II-G.
‡ Average of Series I-G and II-E.

REVIEW OF STUDIES

Having presented the writer's Series D to P and the illuminating new data of the other experimenters, it is helpful to go back, chronologically, to the situation that confronted the writers and, to some extent, the engineering public since the publication of Bazin's volumetric measurements in 1888.

Among the preliminary studies made by the writers, three comparisons may be of some interest.

First Comparison: Six Weirs in Same Channel.—Fig. 8 shows results on a group of six weirs set in the same canal and for which general conditions seemed nearly the same. Among them one might expect close agreement in discharges for the same heads. The averaging lines for the several series show among themselves extreme variations for all heads from 2 to 4 per cent. This is for the "regular" experiments; that is, those made with the regular deflector baffles and stilling-raft arranged to secure a minimum of irregularity in velocities in the channel of approach.

These variations in discharges seem to be rather less than might have been expected if different channels of approach had been used involving the judgments of different men as to sufficiency of baffle systems and proper fineness of finish of crest pieces for correct designation as "sharp-crested."

The reason for these variations in discharges might lie, (1) in the water; or (2) in the weir. In detail, assuming no challenging question as to the general arrangement, dimensions, heads, and discharges, and relying on recent precise investigations, such as Rehbock's, as assurance that effects of temperature changes in the water are exceedingly small at heads above the very lowest, the reasons might lie, (*a*) in velocity conditions in the channel

of approach; or (*b*) in the mechanical conditions at the weir, particularly the sharpness of crest and degree of roughness of the up-stream face.

(*a*).—Examining the distribution of velocities for Series A, B, and C (these are shown on Figs. 33 to 35, and the velocity curves from other series as Figs. 36 to 48, inclusive (see, also, Appendix B (Table 55)), it is seen that the high weir of Series A has an average ratio of about 1.74 between the mean velocity above the crest level and the mean velocity in the whole channel of approach, as compared with an average ratio of about 1.38 for Series B and 1.32 for Series C (the "regular" runs).

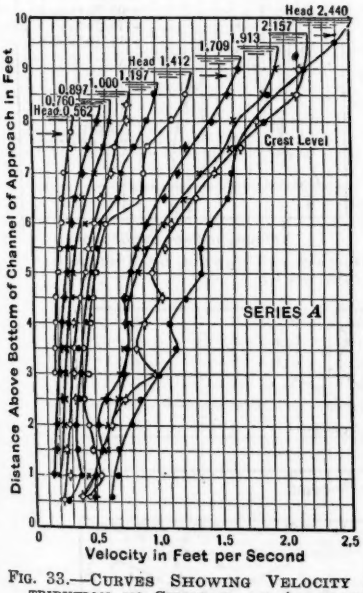

FIG. 33.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES A.

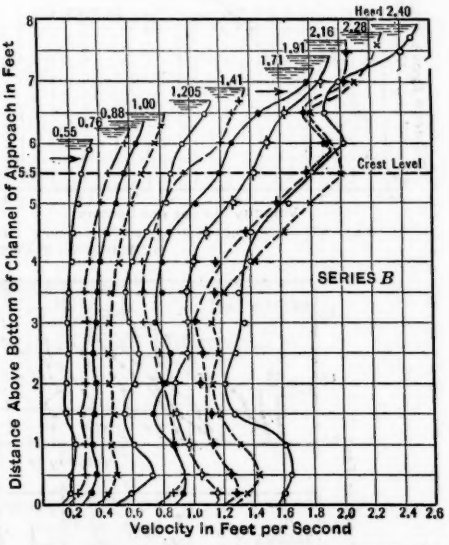

FIG. 34.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES B.

The mean velocities above the crest level, when plotted against the heads, show practically one line for the three series, whereas one would expect somewhat lower velocities for the deeper channel of Series A. (No velocity measurements were taken for Dawson's experiments, but the channel conditions were the same as in Series B.)

By the old formulas, which correct for the velocity of approach, such as those of Fteley and Stearns, or Bazin, a weir 7.5 ft. high would show less discharge than one 5.5 ft. high by an amount about 1.6% for 2.5-ft. head, 0.7% for 1.5-ft. head, 0.4% for 1-ft. head, and 0.1% for 0.5-ft. head. The experiments in Fig. 8 show a decrease only for the highest head as compared with the weirs 5.5 ft. high. This may be due, in small part, to the obstruction offered by the deflector hood to the formation of bottom velocities of normal free-channel magnitude, causing, in effect, flow conditions similar to those for a weir less than 7.5 ft. high in a long canal free from obstructions at the bottom for a much longer distance than 25 ft. up stream.

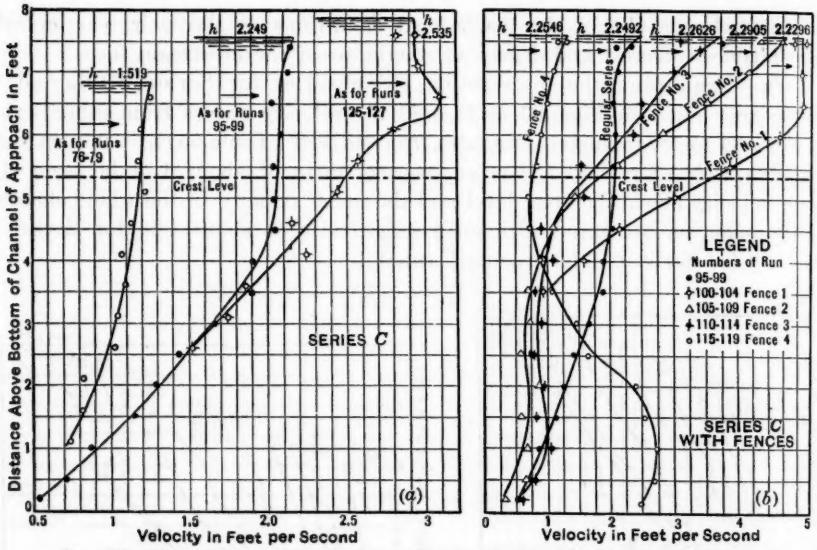

FIG. 35.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES C.

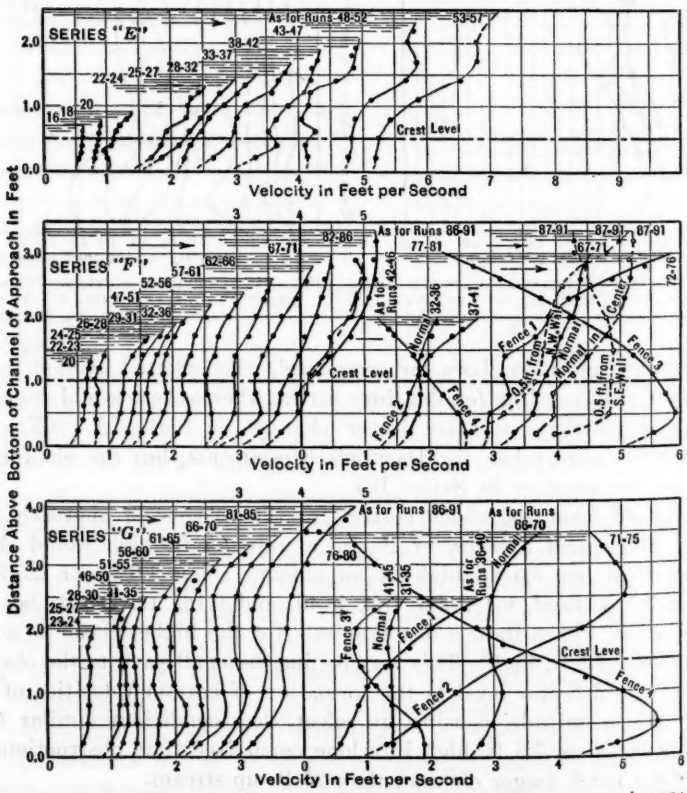

FIG. 36.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES E, F. AND G.

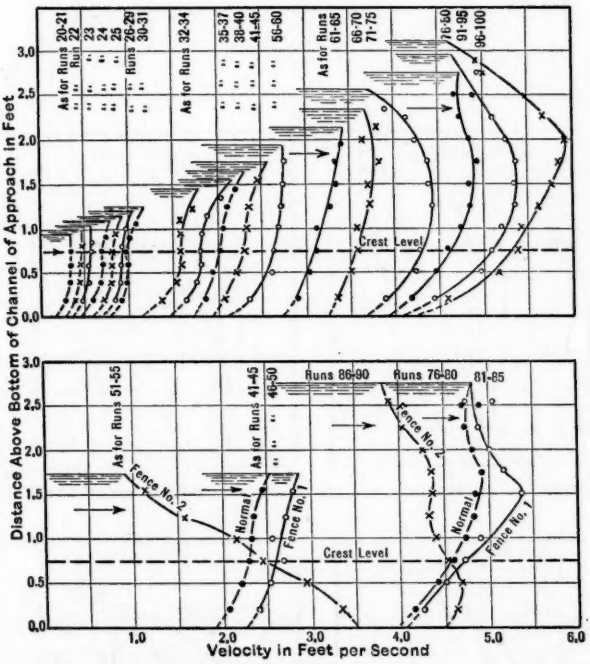

FIG. 37.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES N.

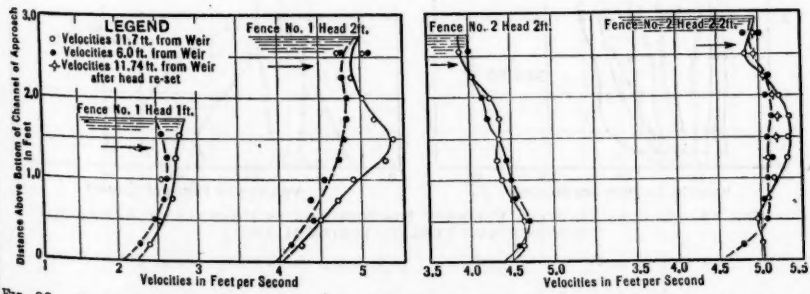
LEGEND
Velocities 11.7 ft. from Weir
Velocities 6.0 ft. from Weir
Velocities 11.74 ft. from Weir after head re-set
Fence No. 1 Head 1ft.
Fence No. 1 Head 2ft.
Distance Above Bottom of Channel of Approach in Feet
Velocities in Feet per Second
Fence No. 2 Head 2ft.
Fence No. 2 Head 2.2ft.
Velocities in Feet per Second

FIG. 38.—CURVES SHOWING COMPARISON OF VELOCITIES AT MID-CHANNEL, AT 11.74 AND 60 FEET UP STREAM FROM WEIR, SCHODER AND TURNER, SERIES N.

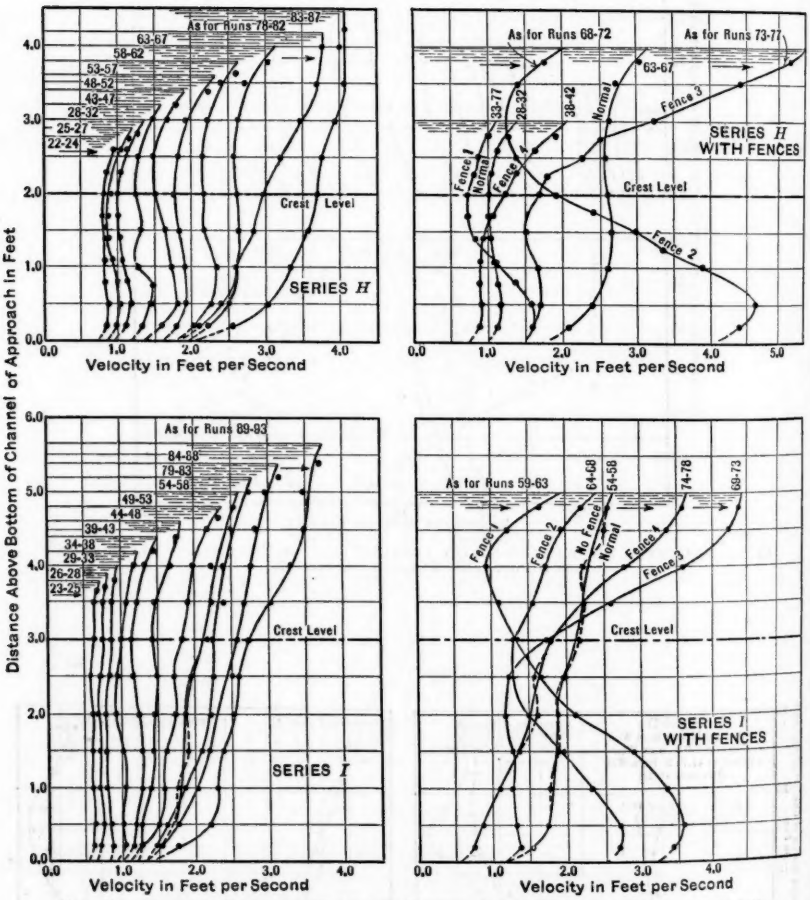

FIG. 39.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES H AND I.

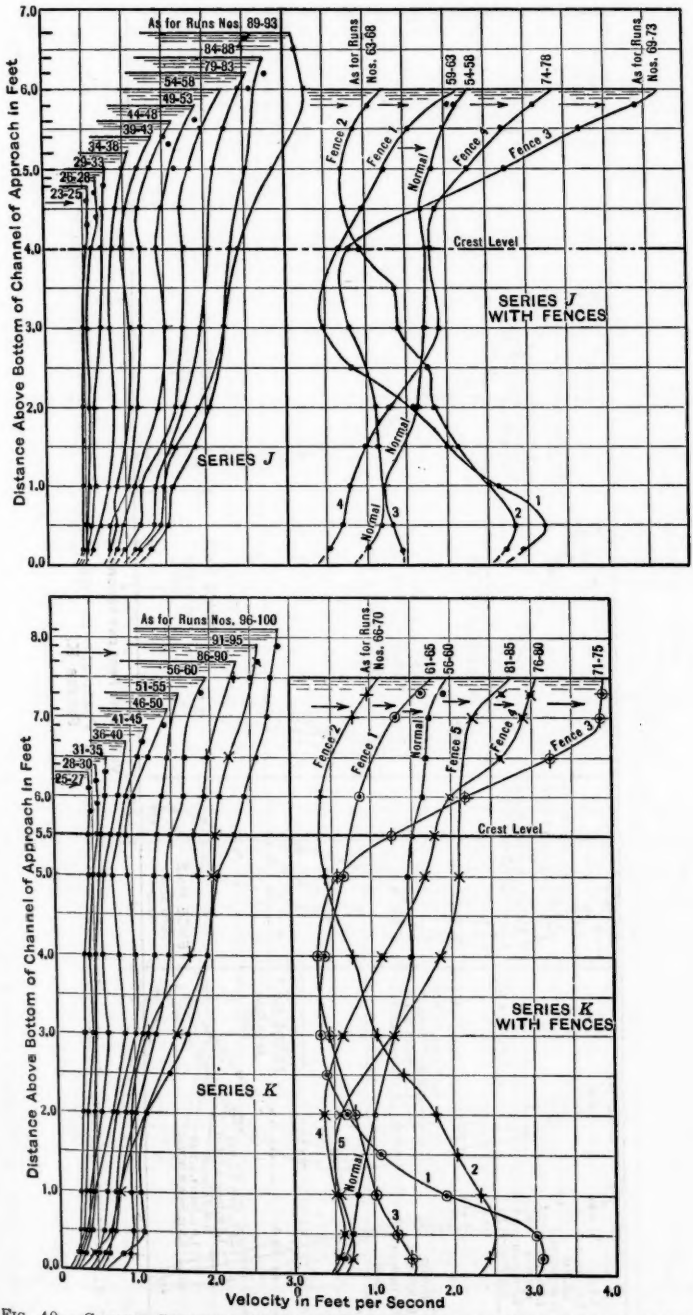

FIG. 40.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNELS OF APPROACH, SCHODER AND TURNER, SERIES J AND K.

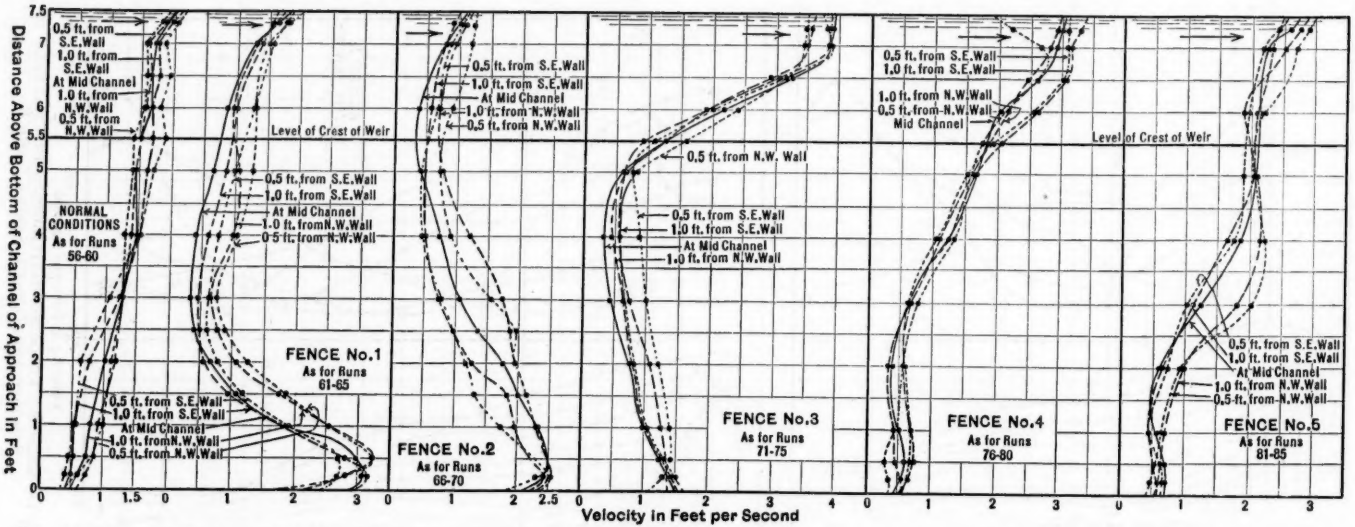

FIG. 41.—CURVES SHOWING VELOCITY DISTRIBUTION ALONG FIVE VERTICALS IN THE SAME CROSS-SECTION (AT GAUGE), SCHODER AND TURNER, SERIES K.

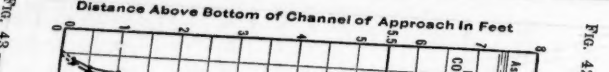

FIG. 43.—
11.74

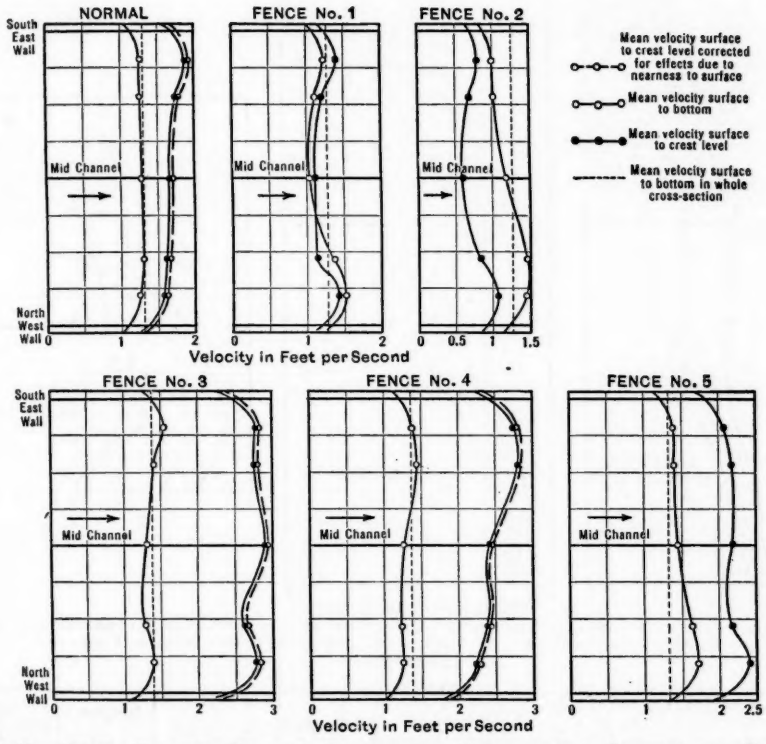
NORMAL
FENCE No. 1
FENCE No. 2
South East Wall
Mid Channel
North West Wall
Mean velocity surface to crest level corrected for effects due to nearness to surface
Mean velocity surface to bottom
Mean velocity surface to crest level
Mean velocity surface to bottom in whole cross-section
Velocity in Feet per Second
FENCE No. 3
FENCE No. 4
FENCE No. 5
Velocity in Feet per Second

FIG. 42.—CURVES SHOWING VELOCITY DISTRIBUTION HORIZONTALLY, SCHODER AND TURNER, SERIES K.

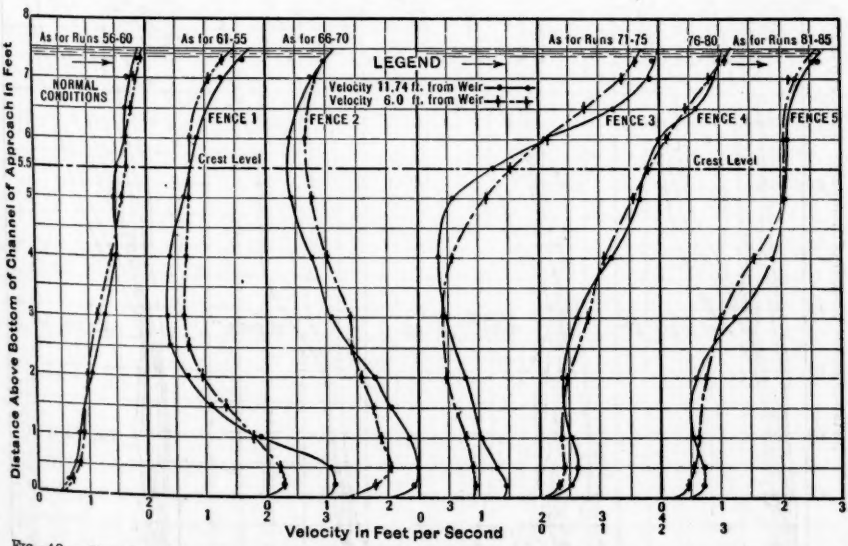

FIG. 43.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION IN MID-CHANNEL AT 11.74 FEET AND 6.0 FEET UP STREAM FROM WEIR, SCHODER AND TURNER, SERIES K.

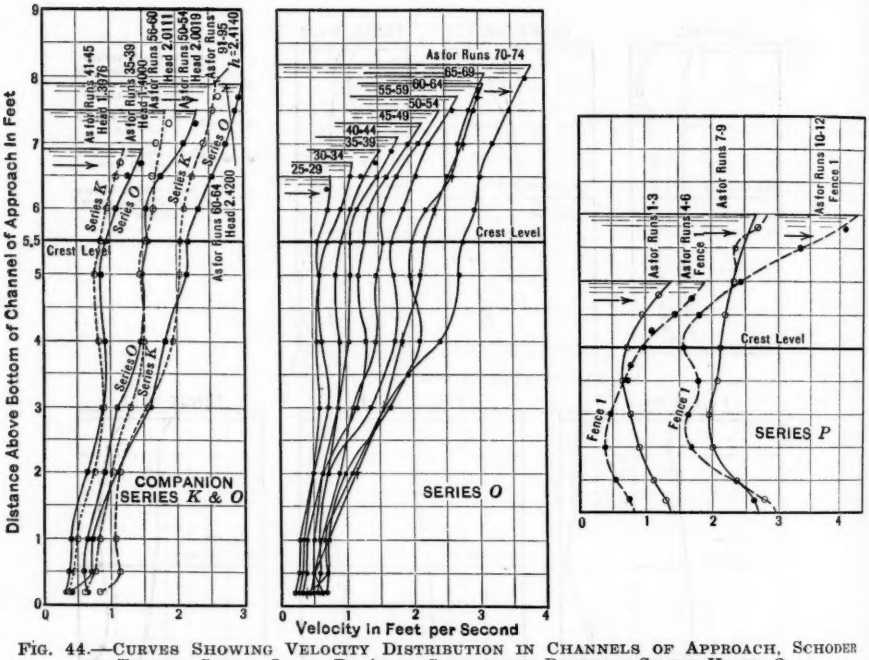

FIG. 44.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNELS OF APPROACH, SCHODER AND TURNER, SERIES O AND P; ALSO, COMPARISON BETWEEN SERIES K AND O AT SAME HEADS.

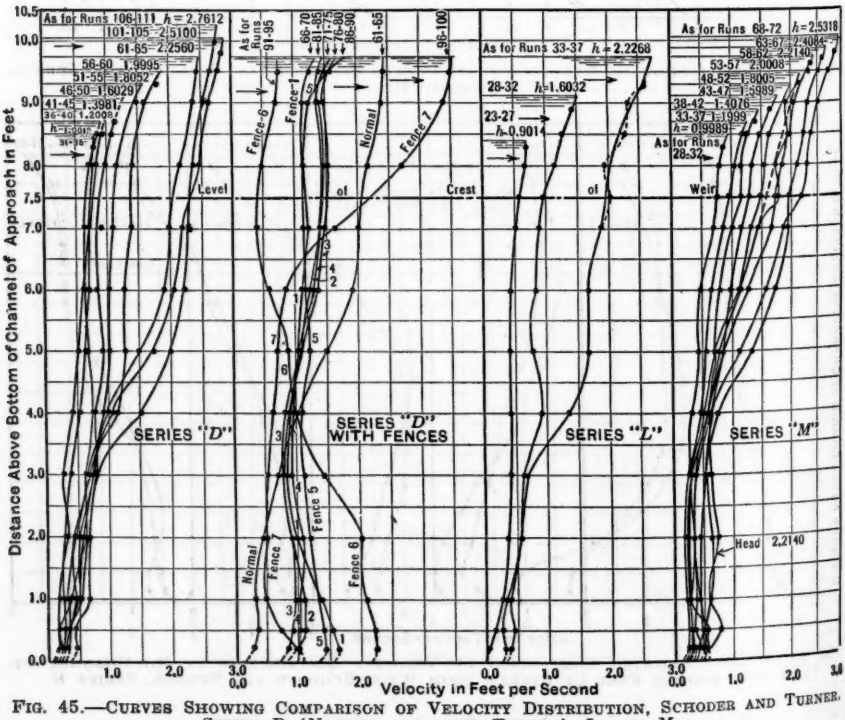

FIG. 45.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION, SCHODER AND TURNER, SERIES D (NORMAL AND WITH FENCES), L, AND M.

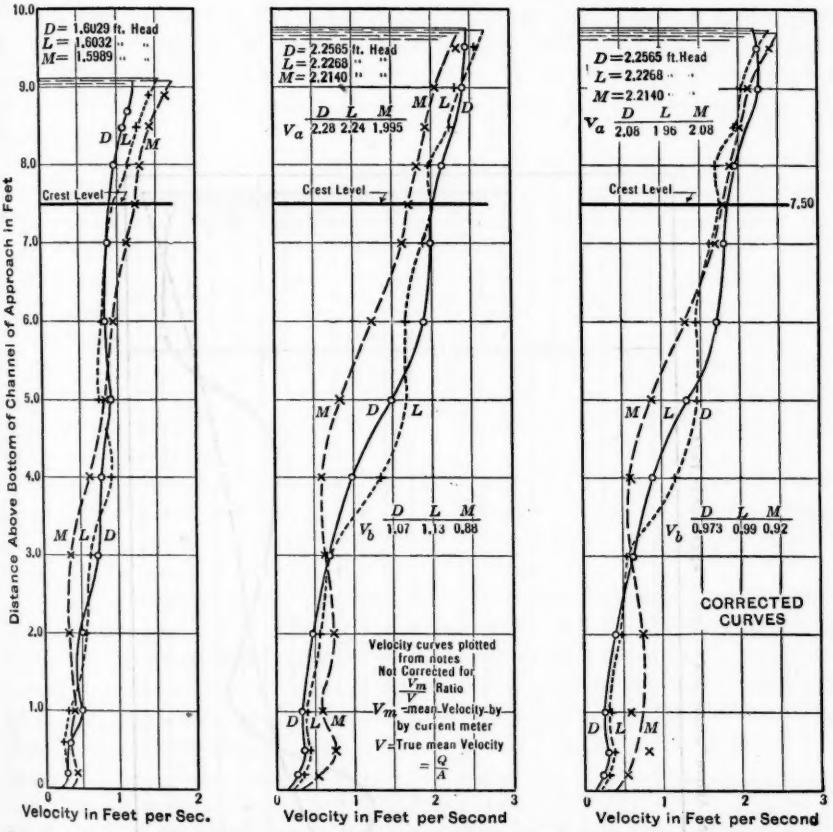
D= 1.6029 ft. Head
L= 1.6032 " "
M= 1.5989 " "
Crest Level
Distance Above Bottom of Channel of Approach in Feet
Velocity in Feet per Sec.
D= 2.2565 ft. Head
L= 2.2268 " "
M= 2.2140 " "
D L M
Va 2.28 2.24 1.995
Crest Level
D L M
Vb 1.07 1.13 0.88
Velocity curves plotted from notes Not Corrected for Vm/V Ratio
Vm =mean Velocity by by current meter
V=True mean Velocity = Q/A
Velocity in Feet per Second
D=2.2565 ft.Head
L=2.2268 " "
M=2.2140 " "
D L M
Va 2.08 1 96 2 08
Crest Level
7.50
D L M
Vb 0.973 0.99 0.92
CORRECTED CURVES
Velocity in Feet per Second

FIG. 46.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION (MID-CHANNEL) FOR SIMILAR HEADS, SCHODER AND TURNER, SERIES D, L, AND M.

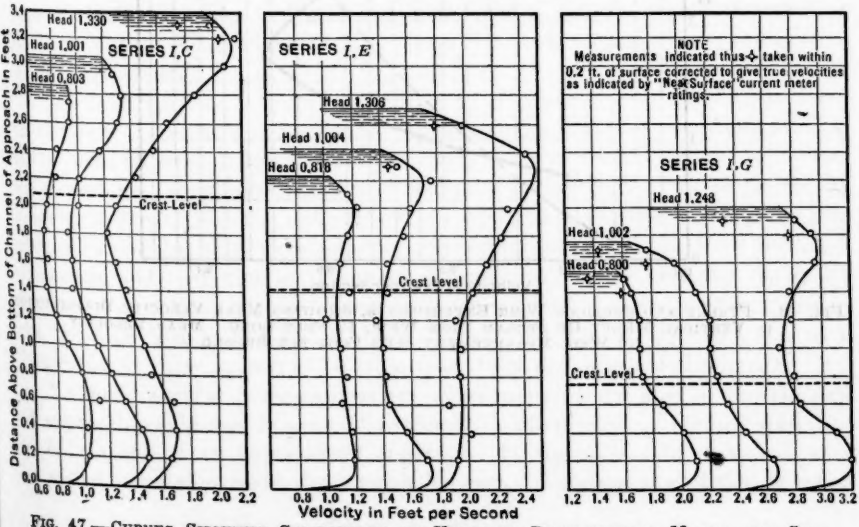

FIG. 47.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION, MEYER AND SEE, SERIES I-C, I-E, AND I-G.

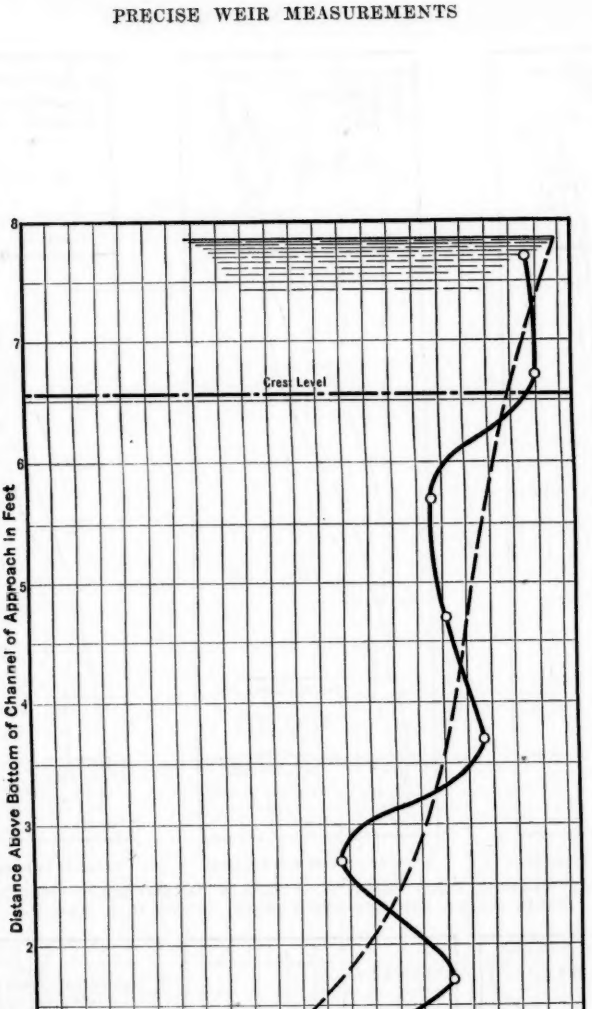

FIG. 48.—FTELEY AND STEARNS' WEIR EXPERIMENTS, SHOWING MEAN VELOCITY DISTRIBUTION IN VERTICAL 5 FEET UP STREAM FROM WEIR, 19 FEET LONG. MEAN VELOCITY BY WEIR MEASUREMENT, 0.63 FOOT PER SECOND.

This effect, however, could not account for the entire lack of conformity. There was a difference in sharpness and smoothness of crest plates, and there was probably a lack of perfect proportionality in the velocity distribution for all heights and heads in a free channel of a length less than four times the highest weir.

(*b*).—Examining the mechanical condition at the weir crests minutely, it is obvious from the descriptions that only two of the weirs, the writers' Series B and Dawson's Series II, deserve to be called strictly sharp-crested; but neither of these two crest pieces had a strictly smooth up-stream face.

It is seen on Fig. 8 that these two series agree within 1% for heads between 0.5 ft. and 2.4 ft.

The considerable effect on discharge due to rusting of an iron crest plate by six weeks' exposure to water in May and June is seen by comparing Dawson's Series II and III. The discharge was increased 1.1, 0.8, 0.75, and 0.6%, respectively, at heads of 0.3, 0.5, 1.0, and 1.5 ft. Above a head of 1.9 ft. the effect seems to be a decrease.

Second Comparison: Old and New Data.—On Fig. 49 are shown the averaging lines for the experimental data (all volumetric measurements on nominally strictly sharp crests) of Fteley and Stearns, Bazin, Martin, the writers' Series B, Dawson's Series II, and Meyer and See.

A cursory inspection of Fig. 49 shows that Bazin's data fall in a group by themselves and indicate distinctly higher discharges than the others. This is true for all heads and heights of weir, as detailed in Table 7. Comparisons have been made with weirs of nearly the same height, except for Column (2), where only the lowest heads are considered. For the other seven comparisons, Bazin's weirs were slightly higher in three cases and slightly lower in four cases.

Other anomalies are apparent. Fteley and Stearns' 1.00-ft. height, at 0.8-ft. head, shows 4.5% excess over the average of Meyer and See's Series 1-F and 2-E of the same height; although at a low head of 0.3 ft., they are in almost exact agreement. The Fteley and Stearns 0.50-ft. weir shows greater discharge at all heads than the Meyer and See 0.41-ft. weir, where the reverse would be expected.

Third Comparison: Effect of Velocity of Approach.—For the 2.25-ft. head in Series C the writers built "fences" to produce in different runs high surface velocities and high bottom velocities, carrying the distribution of velocities up to perhaps the extremes that may be found in practice. Fig. 35 shows the velocity curves. For the same head, an extreme difference of 26% in discharge was produced by variations in the velocity distribution. Data for head, discharge, and velocities are given in Appendix A (Table 11), and Appendix B (Table 55). Even with the lesser re-arrangements of the velocities (such as the "regular" run at the 2.25-ft. head, Series C, compared with the run with Fence No. 3) there is seen to be more than 3.5% difference in discharge.

No doubt careful inspection of the appearance of the flow in the channel of approach would enable cases of extremely high surface or bottom velocities to be noticed by an experienced engineer. It seems certain, however, that no visual inspection could distinguish between discharges that differ as much

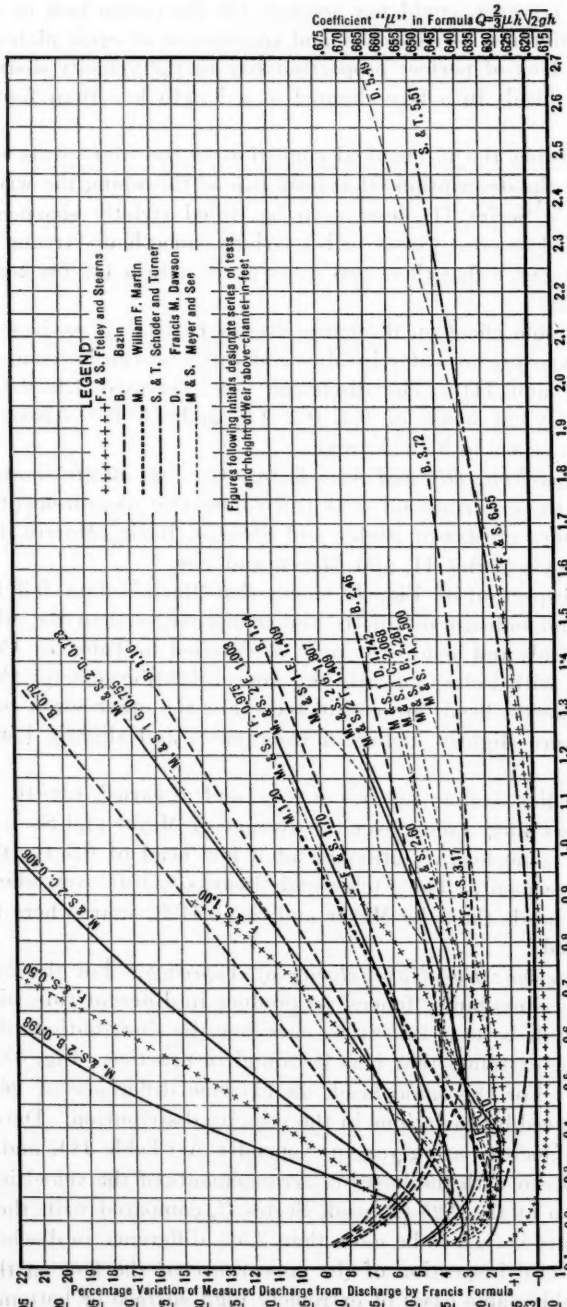
Coefficient "μ" in Formula $Q=\frac{2}{3}\mu h\sqrt{2gh}$
LEGEND
F. & S. Fteley and Stearns
B. Bazin
M. William F. Martin
S. & T. Schoder and Turner
D. Francis M. Dawson
M & S. Meyer and See
Figures following initials designate series of tests and height of weir above channel in feet
Observed Head (h) in Feet
Percentage Variation of Measured Discharge from Discharge by Francis Formula
Coefficient "C" in Formula $Q=CLh^{3/2}$

FIG. 49.—SHARP-CRESTED WEIR EXPERIMENTS. COMPARISON OF EARLIER NEW DATA AND THOSE OF FTELEY AND STEARNS AND BAZIN, ALL REFERRED TO FRANCIS FORMULA.

TABLE 8.—COMPARISON OF DISCHARGE AT VARIOUS HEADS FROM EXPERIMENTS BY VARIOUS EXPERIMENTERS ON SHARP AND NEARLY SHARP-CRESTED WEIRS.

(Fifty-seven series of experiments considered in this tabulation. Height of weir varied from 0.198 ft. to 7.53 ft.)

Series.	Height of weir, in feet.	Observed Head on Weir, in Feet.							
		0.20	0.50	0.80	1.00	1.20	1.50	2.00	2.50
		Percentage of excess of discharge over Francis formula.							
S. and T., A	7.53	2.6	0.9	0.7	0.9	1.3	1.9	2.9	3.9
" , D	7.5	1.3	0.0	0.4	0.7	1.7	3.2	3.1	8.4
" , L	7.5	1.8	0.0	0.2	0.6	1.2	2.2	5.0	...
" , M	7.5	1.2	0.4	0.2	0.8	1.5	2.5	4.0	5.6
Fteley and Stearns	6.55	...	−0.5	−0.3	0.1	0.5	1.1	...	...
S. and T., B	5.51	2.3	0.4	0.2	0.4	0.8	1.5	3.0	4.7
Dawson, I	5.51	5.2	2.5	2.0	2.1	2.3	3.0	4.6	6.4
" , ½-in. round	5.51	9.2	3.4	2.3	2.2	2.5	3.1	4.6	6.4
" , II	5.49	0.4	−0.5	−0.4	−0.2	0.2	1.1	3.4	6.0
" , III	5.49	1.7	0.3	0.3	0.5	1.0	1.7	3.1	...
S. and T., C	5.34	3.5	1.1	1.1	1.5	1.9	2.7	4.3	6.1
" , K	5.50	1.0	0.0	−0.1	0.3	0.6	2.2	4.0	6.9
" , O	5 50	0.3	−0.2	0.2	0.7	1.3	2.5	5.1	7.8
" , J	4.00	1.2	−0.4	−0.2	1.0	1.5	2.8	5.4	7.8
Bazin	3.72	6.5	3.1	2.6	2.7	3.0	3.6	...	...
Fteley and Stearns	3.17	2.5	1.4	2.6	...	...	...	...	...
S. and T., I	3.00	1.1	−0.2	0.4	1.6	2.4	3.8	7.0	9.7
Fteley and Stearns	2.60	...	1.8	3.3	...	...	...	...	...
M. and S., 1-A	2.50	2.6	1.8	2.2	2.6	3.6	...		
Jones, A. B. H	2.50	1.8	0.8	1.6	2.7	4.2	...		
" , C. D. E	2.50	3.4	1.1	1.9	2.9	4.4	...		
" , F	2.50	3.1	1.6	2.1	3.2	4.6	...		
" , G	2.50	5.0	2.4	2.6	3.4	4.7	...		
Weber, II. III, VI	2.50	1.8	1.0	1.9	3.0	4.5	...		
" , IV	2.50	3.2	2.0	2.6	3.7	5.2	...		
" , V	2.50	3.4	2.5	3.3	4.3	5.6	...		
Bazin	2.46	6.7	3.9	4.6	5.4	6.2	...		
M. and S., 1-B	2.287	3.6	2.9	3.2	3.8	4.6	...		
" , 1-C	2.068	3.4	3.7	3.9	4.3	5.2	...		
S. and T., H	2.00	0.5	1.0	2.4	3.6	4.8	7.0		
M. and S., 2 G	1.807	3.7	3.1	4.3	5.7	7.3	...		
" , 1-D	1.742	2.5	2.9	3.4	4.5	5.9	...		
Fteley and Stearns	1.70	3.0	3.1	5.9	...	...	...		

Series.	Height of weir, in feet.	Observed Head on Weir, in Feet.					
		0.20	0.50	0.80	1.00	1.20	1.50
		Percentage excess over Francis formula.					
Bazin	1.64	6.8	5.0	6.7	8.1	9.6	....
S. and T., G	1.50	1.7	2.0	4.3	6.2	7.9	10.7
M. and S., 1-E	1.409	2.1	3.6	4.7	6.5	8.3	....
" 2-F	1.409	1.8	3.1	3.9	5.4	6.9	....
Martin	1.20	2.3	4.0	7.3	9.5	....	....
Bazin	1.16	7.1	6.6	9.6	12.0	14.3	....
M. and S., 2-E	1.003	4.0	4.8	7.2	8.8	10.5	....
Fteley and Stearns	1.00	3.9	6.7	12.0	....	....	....
S. and T., F	1.00	2.0	4.2	8.0	10.5	13.1	17.0
M. and S., 1-F	0.975	3.1	5.4	7.8	9.2	10.6	....
Bazin	0.79	7.4	9.1	13.4	16.5	19.5	....
M. and S., 1-G	0.755	2.6	7.4	10.5	13.1	15.8	....
S. and T., N	0.75	2.6	5.8	11.0	14.3	17.0	22.0
M. and S., 2-D	0.723	3.2	6.5	10.5	13.3	16.1	....
Fteley and Stearns	0.50	6.4	13.9	....	....	....	....
S. and T., E	0.50	5.2	10.6	18.4	25.0	28.6	37.6
M. and S., 2-C	0.406	4.4	11.6	18.6	22.3	....	....
" , 2-B	0.198	5.5	18.5	27.2	....	....	....

as 4 or 5% at the same head due solely to different distributions of velocities in the channel of approach.

The writers, in view of such conflicts and variations, and needing as reliable an estimate as possible for the discharge over a large reference weir already used by them in other investigations, decided to undertake the studies covered by Series D to P as an auxiliary work.

General Comparison of Old and New Data.—In Table 8 is given a fairly comprehensive comparison of discharges at various heads (0.2-ft. to 2.5-ft.) by various experimenters (old and new) on weirs of various heights, all weirs of the sharp or nearly sharp-crested type. The data are given in terms of the percentage departure from discharges calculated by the simple Francis formula,

$$Q = 3.33\ L\ h^{\frac{3}{2}}, \text{ or } Q = 0.623 \times \frac{2}{3}\ L\ h\ \sqrt{2\,g\,h}$$

All values of the percentage variation are taken from the diagrams, averaging the plotted points, except in the case of the writers' Series D, I, J, K, L, M, N, and O, for which the data are from the actual measurements or interpolated between measurements.

Perhaps this comparison serves best to emphasize the present state of weir hydraulics when refinements of one sort or another are disregarded.

Conclusions Indicated

Supplementary to the main findings given in the Synopsis and to the conclusions stated in the text, or made obvious by tables and diagrams, a few other deductions are of interest.

Condition of Crest Piece.—The data clearly show that both the roughness of the up-stream face and the rounding of the up-stream top corner of the crest plate increase the discharge for the same head, over that for smooth, sharp conditions. A combination of the two effects each in slight degree, could easily amount to from 1 to 2 per cent.

In tests and routine measurements demanding the utmost precision it seems quite necessary to have the crest plate very smooth in order to avoid uncertainties as to the degree of roughness. Whether it is preferable to try to maintain a sharp corner or one definitely rounded to a very small radius may be as much a practical as a hydraulic question. A difficulty with a slightly rounded corner is to have it exactly to dimension and form, but this would not apply to a rounding to, say, $\frac{3}{16}$ or $\frac{1}{4}$ in. radius.

Velocity Distribution in the Channel of Approach.—It seems hardly sufficient to state, as did Bazin, that a certain precision is attainable "provided, however, that the arrangements of our standard weir are exactly reproduced", unless it is indicated clearly which of the arrangements are essential to the "standard weir", which are to be minutely copied, and which are left to the necessities of the particular case or to the judgment of the engineer.

The velocity conditions in the channel of approach should be standardized to some degree for those cases where control is possible. If this were done the use of a formula involving only the head, height, and length of weir

would probably be permissible. A uniform distribution of velocities is not a natural condition for an open channel. Then, too, a measuring weir can seldom be placed conveniently at the down-stream end of a long canal. In practice the demand is for the shortest possible canal. Often, also, the entry conditions are turbulent, sometimes violently so. The use of baffles seems essential.

In many cases it would be out of the question to depend entirely on the length of the channel of approach to re-arrange the velocities. The measurements of Fteley and Stearns and of the writers indicate that a particular distribution, once started, persists surprisingly in a deep channel. For low weirs with heads higher than the weir, and, consequently, with high velocities of approach, the evidence shows a regularity of a high order as far as discharge is concerned. Figs. 44 and 46 show the differences found by the writers in channels nominally the same.

Measurements of Head.—The side-wall piezometer connected to a stilling-well seems to be an excellent device. However, in addition to the usual precautions concerning perpendicular boring of the hole and avoidance of projections into the channel, the matter of freedom from centrifugal and suction effects due to cross-currents is of some consequence. In recent construction the stilling-well has been connected symmetrically to a pair of opposite side-wall piezometers.

The writers have found the float-gauge, as described, very convenient. A separate setting for each reading is avoided and all the surges can be noted. The use of a glass stilling-pipe allows precise work with a hook-gauge or point-gauge with visibility from below the water surface for the former.

The plumb-bob gauge has the advantages of simplicity, ease of construction, and direct contact with the water surface in the channel of approach. However, it requires a separate setting for each reading.

Friction in the Channel of Approach.—Ordinarily, the effect of friction between the point where the head is measured and the weir is negligibly small; but in an extreme case, as for the writers' Series E (2-ft. head on a weir 0.5 ft. high), the surface fall in 12 ft. for open-channel flow amounts to nearly 1% of the head, and the decrease of discharge for this case is about 1.5% because of friction. For a 1-ft. head the effect on the discharge is still 1 per cent.

For very narrow channels, Martin's experiments indicate that allowances for friction, by open channel laws, between the head gauge station and the weir, are adequate subtractive corrections for the head. The writers' large-scale experiments furnish more evidence on this point.

Acknowledgments

It is not intended to imply that there may not long have been available to a few engineers and scientists working information on the points mentioned in the paper. The writers know that all the points specified in the title have been the subjects of not a little consideration and some experimentation previously. To some extent due acknowledgment is made in the paper, but it is undoubtedly inadequate and incomplete. The writers would welcome additional prior or contemporaneous evidence.

The completion of this paper has been greatly assisted by grants from the Heckscher Research Foundation of Cornell University in 1922, 1923, and 1925. Professor Francis M. Dawson was Research Assistant under these grants. The great aid rendered by him in the preparation of the paper is appreciatively and gratefully acknowledged.

(Professor Schoder wishes to record his profound regret and sense of a great loss to engineering science in that Professor Turner's life was cut short before the completion of these studies for publication; and yet it was a great privilege to enjoy the long scientific comradeship and deep friendship that existed between the writers as partners during the strenuous experimentation involved in these and the other (still unpublished) researches.)

APPENDIX A

TABLE 9.—SCHODER AND TURNER, (NEARLY) SHARP-CRESTED WEIRS, SERIES A.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0494	0.0502	0.1767	0.0498	0.0419	13.2	34	1.1954	1.1968	18.597			
2	0.0604	0.0612	0.2297	0.0608	0.0544	9.0	35	1.1954	1.1964	18.743			
3	0.0773	0.0793	0.3351	0.0783	0.0794	8.9	36	1.1963	1.1968	18.715	1.1968	4.4249	1.5
4	0.0904	0.0910	0.4104	0.0907	0.0973	6.9	37	1.1968	1.1981	18.711			
5	0.1148	0.1162	0.5837	0.1155	0.1383	5.7	38	1.1965	1.1993	18.765			
6	0.1546	0.1557	0.8940	0.1552	0.2118	4.0	39	1.4114	1.4111	23.885			
7	0.1788	0.1802	1.1090	0.1795	0.2607	3.0	40	1.4106	1.4116	23.953			
8	0.1978	0.1980	1.2674	0.1979	0.3003	2.4	41	1.4110	1.4121	23.974	1.4115	5.6642	1.4
9	0.2529	0.2553	1.8367	0.2541	0.4352	2.0	42	1.4121	1.4120	23.980			
10	0.2848	0.2854	2.1800	0.2851	0.5166	1.9	43	1.7043	1.7063	32.099			
11	0.3206	0.3212	2.5913	0.3209	0.6141	1.5	44	1.7082	1.7096	32.254			
12	0.3717	0.3721	3.2277	0.3719	0.7649	1.3	45	1.7098	1.7108	32.243	1.7088	7.6223	2.5
13	0.3959	0.3963	3.5491	0.3961	0.8410	1.3	46	1.7100	1.7108	32.275			
14	0.4209	0.4211	3.8877	0.4210	0.9213	1.3	47	1.7094	1.7085	32.248			
15	0.4672	0.4682	4.5463	0.4677	1.0773	1.1	48	1.9090	1.9117	38.428			
16	0.4972	0.4984	4.9739	0.4978	1.1787	0.8	49	1.9124	1.9147	38.530			
17	0.5486	0.5501	5.7340	0.5502	1.3641	0.4	50	1.9124	1.9147	38.626	1.9133	9.1237	3.5
18	0.5499	0.5520	5.7790				51	1.9144	1.9148	38.703			
19	0.7590	0.7612	9.3633				52	1.9144	1.9148	38.647			
20	0.7598	0.7610	9.3996				53	2.1612	.....	45.908			
21	0.7597	0.7610	9.4155	0.7602	2.2216	0.7	54	2.1532	.....	46.030			
22	0.7596	0.7598	9.3993				55	2.1612	.....	46.247	2.1572	10.885	3.2
23	0.7599	0.7608	9.4082				56	2.1532	.....	46.108			
24	0.8788	0.8800	11.670				57	2.1572	.....	46.076			
25	0.8788	0.8800	11.718				58	2.2767	.....	49.900	2.2767	11.784	3.0
26	0.8790	0.8800	11.716	0.8793	2.7645	0.7	59	2.4332	.....	55.911			
27	0.8786	0.8793	11.743				60	2.4502	.....	56.253			
28	0.8788	0.8800	11.633				61	2.4437	.....	55.440	2.4398	13.150	3.6
29	0.9986	0.9993	14.177				62	2.4327	.....	55.593			
30	0.9988	0.9991	14.227				63	2.4392	.....	55.436			
31	0.9982	0.9992	14.212	0.9989	3.3607	1.1							
32	0.9992	0.9990	14.232										
33	0.9982	0.9987	14.203										

TABLE 10.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES B.

No. of run.	Observed Head, in Feet.		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-Gauge.	Plumb-bob Gauge.						Float-Gauge.	Plumb-bob Gauge.				
1	0.0496	0.0510	0.1778	0.0504	0.0414	10.6	39	1.4088	.....	23.904			
2	0.0499	0.0509	0.1743				40	1.4218	.....	24.164			
3	0.0636	0.0639	0.2567	0.0638	0.0608	13.2	41	1.4210	.....	24.171	1.4174	5.7085	1.6
4	0.0774	0.0780	0.3621	0.0777	0.0858	19.0	42	1.4176	.....	24.134			
5	0.0969	0.0986	0.4693	0.0978	0.1112	9.1	43	1.4193	.....	24.115			
6	0.1169	0.1179	0.6020	0.1174	0.1427	6.6	44	1.4160	.....	24.050			
7	0.1379	0.1393	0.7613	0.1386	0.1804	5.0	45	1.7061	.....	31.783			
8	0.1648	0.1659	0.9777	0.1654	0.2317	3.4	46	1.7084	.....	31.918			
9	0.1830	0.1845	1.1358	0.1838	0.2691	2.6	47	1.7096	.....	31.840	1.7088	7.5590	1.6
10	0.2032	0.2039	1.3193	0.2036	0.3126	2.2	48	1.7098	.....	32.024			
11	0.2488	0.2499	1.7813	0.2494	0.4221	1.8	49	1.7101	.....	31.980			
12	0.3018	0.3019	2.3542	0.3018	0.5579	1.1	50	1.9128	.....	37.975			
13	0.3526	0.3524	2.9730	0.3525	0.7045	1.1	51	1.9033	.....	37.848			
14	0.3983	0.3994	3.5537	0.3988	0.8421	0.4	52	1.9098	.....	38.142	1.9092	9.0018	2.5
15	0.4520	0.4525	4.2910	0.4522	1.0168	0.4	53	1.9081	.....	37.917			
16	0.4923	0.4925	4.8860	0.4952	1.1683	0.7	54	1.9118	.....	38.055			
17	0.4979	0.4981	4.9744				55	2.1480	.....	45.777			
18	0.5519	0.5527	5.7796	0.5523	1.3696	0.2	56	2.1536	.....	45.914			
19	0.7536	0.7549	9.1935				57	2.1530	.....	45.861	2.1513	10.865	3.4
20	0.7522	0.7537	9.1630				58	2.1520	.....	45.834			
21	0.7516	0.7537	9.1428	0.7528	2.1698	— 0.02	59	2.1498	.....	45.871			
22	0.7517	0.7537	9.2145				60	2.2758	.....	50.256			
23	0.7509	0.7519	9.0698				61	2.2673	.....	50.237			
24	0.8700	.....	11.449				62	2.2688	.....	49.958	2.2698	11.897	4.5
25	0.8720	.....	11.499				63	2.2725	.....	50.360			
26	0.8721	.....	11.539	0.8719	2.7291	0.7	64	2.2648	.....	50.226			
27	0.8732	.....	11.555				65	2.4346	.....	55.783			
28	0.8724	.....	11.541				66	2.4426	.....	55.737			
29	1.0011	.....	14.137				67	2.4336	.....	56.005	2.4393	13.256	4.5
30	1.0015	.....	14.185				68	2.4359	.....	55.721			
31	1.0002	.....	14.193	1.0020	3.3655	0.8	69	2.4496	.....	56.462			
32	1.0036	.....	14.232										
33	1.0037	.....	14.265										
34	1.2076	.....	18.727										
35	1.2068	.....	18.743										
36	1.2066	.....	18.784	1.2079	4.4518	0.7							
37	1.2064	.....	18.781										
38	1.2120	.....	18.898										

TABLE 11.—SCHODER AND TURNER, (NEARLY) SHARP-CRESTED WEIRS, SERIES C.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0363	0.0366	0.0850	0.0364	0.0201	—13.0
2	0.0384	0.0384	0.0990	0.0384	0.0235	— 6.4
3	0.0418	0.0420	0.1175	0.0419	0.0278	— 2.8
4	0.0443	0.0447	0.1302	0.0445	0.0309	— 1.0
* 5	0.0497	0.0487	0.1603	0.0489	0.0383	+ 6.4
* 6	0.0482	0.0489	0.1630			
7	0.0536	0.0538	0.1920	0.0537	0.0455	9.9
* 8	0.0553	0.0549	0.1990	0.0551	0.0472	9.5
* 9	0.0612	0.0615	0.2107	0.0612	0.0571	13.3
* 10	0.0611	0.0612	0.2413			
* 11	0.0723	0.0719	0.3212	0.0721	0.0763	18.5
* 12	0.0720	0.0722	0.3230			
13	0.0810	0.0816	0.3800	0.0813	0.0900	16.6
* 14	0.0815	0.0831	0.3987	0.0827	0.0945	19.3
* 15	0.0829	0.0833	0.3993			
16	0.0837	0.0832	0.4077	0.0834	0.0966	20.4
17	0.0842	0.0837	0.4157	0.0840	0.0985	21.5
18	0.0857	0.0849	0.4113	0.0853	0.0975	17.5
19	0.0877	0.0878	0.4243	0.0878	0.1005	16.1
20	0.0879	0.0879	0.4197	0.0879	0.0995	14.6
21	0.0885	0.0878	0.4383	0.0882	0.1039	19.2
* 22	0.0904	0.0894	0.4573	0.0902	0.1088	20.6
* 23	0.0910	0.0899	0.4610			
24	0.0910	0.0914	0.4390	0.0912	0.1040	13.4
25	0.0920	0.0919	0.4740	0.0920	0.1123	20.9
26	0.0926	0.0928	0.4593	0.0927	0.1088	15.7
27	0.0933	0.0935	0.4483	0.0934	0.1062	11.8
28	0.0936	0.0937	0.4777	0.0936	0.1132	18.7
29	0.0943	0.0935	0.4700	0.0939	0.1114	16.3
30	0.0964	0.0962	0.5096	0.0963	0.1208	21.4
* 31	0.1013	0.1004	0.4993	0.1014	0.1183	10.0
* 32	0.1029	0.1010	0.4990			
33	0.1018	0.1022	0.4947	0.1020	0.1172	8.0
34	0.1025	0.1024	0.5077	0.1024	0.1203	10.3
35	0.1082	0.1071	0.5413	0.1076	0.1283	9.1
36	0.1087	0.1079	0.5367	0.1083	0.1272	7.2
37	0.1092	0.1096	0.5530	0.1094	0.1310	8.8
38	0.1117	0.1114	0.5692	0.1116	0.1349	8.7
39	0.1156	0.1157	0.5870	0.1156	0.1391	6.3
* 40	0.1194	0.1194	0.6217	0.1196	0.1477	7.3
* 41	0.1196	0.1198	0.6247			
* 42	0.1382	0.1370	0.7573	0.1378	0.1794	5.3
* 43	0.1391	0.1389	0.7570			
44	0.1440	0.1436	0.7900	0.1438	0.1872	3.1
45	0.1616	0.1605	0.9483	0.1609	0.2249	4.7
46	0.1606	0.1608	0.9500			
47	0.1800	0.1797	1.1163	0.1801	0.2646	4.0
48	0.1801	0.1805	1.1173			
49	0.2006	0.1997	1.3029	0.2002	0.3091	3.6
50	0.2004	0.2003	1.3058			
51	0.2514	0.2507	1.8139	0.2510	0.4293	2.5
52	0.2513	0.2508	1.8094			
53	0.2986	0.2985	2.3433	0.2988	0.5567	2.4
54	0.2990	0.2982	2.3555			
55	0.3501	0.3503	2.9547	0.3501	0.7010	1.6
56	0.3502	0.3499	2.9617			
57	0.3996	0.3991	3.5900	0.3992	0.8518	1.4
58	0.3993	0.3990	3.5990			
59	0.4502	0.4496	4.2850	0.4500	1.0177	1.2
60	0.4501	0.4500	4.3040			
61	0.5024	0.5023	5.0571	0.5026	1.2000	1.1
62	0.5029	0.5027	5.0708			
63	0.5995	0.5985	6.5624	0.5987	1.5585	1.0
64	0.5990	0.5977	6.5914			
65	1.0000	.....	14.270	1.0006	3.3831	1.5
66	1.0004	.....	14.285			
67	1.0014	.....	14.275			
68	1.2062	.....	18.953	1.2058	4.5005	2.1
69	1.2064	.....	19.031			
70	1.2048	.....	18.993			
71	1.4034	.....	23.899	1.3994	5.6361	2.2
72	1.4014	.....	23.843			
73	1.3986	.....	23.820			
74	1.3958	.....	23.624			
75	1.3976	.....	23.735			
76	1.5964	.....	29.118	1.5897	6.8664	2.9
77	1.5900	.....	28.956			
78	1.5884	.....	28.964			
79	1.5863	.....	28.866			
80	1.8034	.....	35.022	1.7947	8.2805	3.4
81	1.7939	.....	34.944			
82	1.7933	.....	34.951			
83	1.7906	.....	34.914			
84	1.7921	.....	34.887			
85	2.0030	.....	41.622	1.9983	9.8224	4.4
86	1.9996	.....	41.467			
87	1.9996	.....	41.462			
88	1.9966	.....	41.374			
89	1.9928	.....	41.327			
90	2.1976	.....	48.510	2.1987	11.427	5.6
91	2.1941	.....	48.317			
92	2.1956	.....	48.264			
93	2.1894	.....	47.970			
94	2.1918	.....	48.038			
95	2.2475	.....	49.778	2.2442	11.750	5.0
96	2.2446	.....	49.720			
97	2.2439	.....	49.535			
98	2.2411	.....	49.303			
99	2.2438	.....	49.595			
100	2.2415	.....	59.996	2.2427	14.211 (Fence 1)	27.1
101	2.2434	.....	60.072			
102	2.2447	.....	60.332			
103	2.2405	.....	59.800			
104	2.2432	.....	59.646			
105	2.2614	.....	55.652	2.2711	13.312 (Fence 2)	16.8
106	2.2591	.....	55.866			
107	2.2604	.....	55.862			
108	2.2876	.....	56.682			
109	2.2871	.....	56.811			
110	2.2661	.....	52.064	2.2630	12.315 (Fence 3)	8.6
111	2.2611	.....	51.632			
112	2.2614	.....	51.943			
113	2.2644	.....	52.161			
114	2.2619	.....	52.056			
115	2.2679	.....	48.506	2.2670	11.490 (Fence 4)	1.1
116	2.2651	.....	48.311			
117	2.2674	.....	48.508			
118	2.2668	.....	48.630			
119	2.2676	.....	48.483			
120	2.3737	.....	54.175	2.3729	12.823	5.3
121	2.3776	.....	54.180			
122	2.3751	.....	54.157			
123	2.3684	.....	54.155			
124	2.3698	.....	53.899			
125	2.5312	.....	60.011	2.5304	14.237	6.2
126	2.5282	.....	60.089			
127	2.5318	.....	60.144			

* Runs made with old type float.

TABLE 12.—Schoder and Turner, Sharp-Crested Weirs, Series D.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0284	0.0279	0.0740	0.0282	0.0175	10.8
2	0.0315	0.0317	0.0910	0.0316	0.0216	15.5
3	0.0350	0.0349	0.1040	0.0350	0.0246	12.8
4	0.0395	0.0391	0.1197	0.0393	0.0284	9.7
5	0.0459	0.0459	0.1515	0.0459	0.0359	9.5
6	0.0525	0.0523	0.1830	0.0524	0.0434	8.8
7	0.0601	0.0605	0.2207	0.0603	0.0523	6.1
8	0.0677	0.0673	0.2623	0.0675	0.0622	6.5
9	0.0771	0.0769	0.3163	0.0770	0.0750	5.3
10	0.0873	0.0867	0.3762	0.0870	0.0891	4.2
11	0.0987	0.0979	0.4515	0.0983	0.1070	4.3
12	0.1145	0.1145	0.5617	0.1145	0.1331	3.2
13	0.1349	0.1345	0.7110	0.1347	0.1685	2.4
14	0.1559	0.1555	0.8782	0.1557	0.2081	1.7
15	0.1802	0.1807	1.0922	0.1804	0.2588	1.5
16	0.2003	0.2005	1.2773	0.2004	0.3027	1.3
17	0.2501	0.2493	1.7683	0.2497	0.4190	0.8
18	0.3019	0.3015	2.3167	0.3017	0.5561	0.8
19	0.3511	0.3510	2.9413	0.3510	0.6970	0.6
20	0.4005	0.4009	3.5747	0.4007	0.8471	0.3
21	0.4539	0.4533	4.3083	0.4536	1.0209	0.4
22	0.5035	0.5029	5.0138	0.5031	1.1885	0.0
23	0.5033	0.5028	5.0175			
24	0.8061	0.8073	10.258			
25	0.8061	0.8059	10.184	0.8066	2.4218	0.4
26	0.8060	0.8069	10.219			
27	0.8072	0.8073	10.221			
28	0.9001	0.8986	12.052			
29	0.8993	0.8989	12.058	0.8990	2.8559	0.6
30	0.8988	0.8986	12.045			
31	1.0036	1.0037	14.216			
32	1.0035	1.0046	14.214			
33	1.0029	1.0041	14.240	1.0039	3.3782	0.7
34	1.0031	1.0051	14.256			
35	1.0035	1.0045	14.249			
36	1.2046	1.2068	18.862			
37	1.2061	1.2071	18.885			
38	1.2063	1.2059	18.974	1.2066	4.4865	1.7
39	1.2081	1.2103	18.962			
40	1.2072	1.2089	18.984			
41	1.3969	1.3966	23.803			
42	1.3954	1.3951	23.852			
43	1.3964	1.3971	23.864	1.3967	5.6479	2.8
44	1.3972	1.3961	23.858			
45	1.3974	1.3986	23.794			
46	1.5951	1.5928	29.322			
47	1.5952	1.5954	29.359			
48	1.5932	1.5964	29.277	1.5945	6.9455	3.6
49	1.5961	1.5951	29.350			
50	1.5939	1.5914	29.243			
51	1.8047	1.8073	34.883			
52	1.7992	1.8055	34.865			
53	1.8020	1.8072	34.854	1.8046	8.2784	2.5
54	1.8058	1.8050	35.088			
55	1.8050	1.8048	34.986			
56	2.0030	2.0016	40.889			
57	1.9985	2.0018	40.989			
58	2.0021	1.9998	41.047	2.0015	9.7185	3.1
59	2.0020	2.0007	41.161			
60	2.0034	2.0017	40.975			
61	2.2678	2.2734	50.974			
62	2.2829	2.2792	51.205			
63	2.2451	2.2396	50.284	2.2582	12.002	6.2
64	2.2391	2.2381	50.138			
* 65	2.2449	2.2469	48.965			
66	2.2498	2.2540	47.723			
67	2.2497	2.2564	47.967			
68	2.2556	2.2554	48.122	2.2539	11.372 (Fence 1)	0.9
69	2.2555	2.2563	48.146			
70	2.2518	2.2544	47.983			
71	2.2479	2.2520	48.050			
72	2.2478	2.2502	47.994			
73	2.2471	2.2478	48.174	2.2474	11.402 (Fence 2)	1.6
74	2.2462	2.2462	48.041			
75	2.2469	2.2414	48.317			
76	2.2515	2.2522	48.791			
77	2.2512	2.2531	48.614			
78	2.2535	2.2508	48.685	2.2528	11.551 (Fence 3)	2.6
79	2.2552	2.2508	48.764			
80	2.2536	2.2556	48.872			
81	2.2520	2.2533	48.481			
82	2.2484	2.2187	48.397			
83	2.2471	2.2488	48.340	2.2501	11.472 (Fence 4)	2.1
84	2.2526	2.2516	48.402			
85	2.2480	2.2506	48.433			
86	2.2473	2.2510	47.984			
87	2.2456	2.2523	48.009			
88	2.2465	2.2492	47.977	2.2484	11.373 (Fence 5)	+1.3
89	2.2455	2.2503	47.895			
90	2.2465	2.2502	48.095			
91	2.2476	2.2554	47.173			
92	2.2432	2.2546	47.063			
93	2.2448	2.2570	47.099	2.2503	11.151 (Fence 6)	−0.8
94	2.2464	2.2554	47.037			
95	2.2431	2.2555	46.922			
96	2.2535	2.2561	51.094			
97	2.2518	2.2456	51.281			
98	2.2499	2.2499	50.730	2.2510	12.071 (Fence 7)	+7.3
99	2.2520	2.2521	50.685			
100	2.2468	2.2519	50.912			
101	2.5062	2.5082	60.796			
102	2.5106	2.5077	60.432			
103	2.5045	2.5037	60.106	2.5057	14.320	8.4
104	2.5012	2.5014	60.452			
105	2.5092	2.5044	60.357			
106	2.7412	2.7399	68.648			
107	2.7288	2.7231	67.324			
108	2.7602	2.7581	69.270	2.7490	16.234	7.0
109	2.7599	2.7555	69.131			
110	2.7569	2.7536	68.502			
111	2.7598	2.7515	68.181			

* This run not used in averages.

TABLE 13.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES E.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0288	0.0287	0.0765	0.0288	0.0181	11.7	33	1.2072	.....	24.126			
2	0.0350	0.0334	0.1043	0.0342	0.0247	17.1	34	1.2095	.....	24.198			
3	0.0382	0.0373	0.1160	0.0378	0.0275	12.2	35	1.2055	.....	24.092	1.2090	5.7344	29.5
4	0.0424	0.0418	0.1357	0.0421	0.0322	11.8	36	1.2112	.....	24.240			
5	0.0514	0.0508	0.1763	0.0511	0.0418	8.6	37	1.2114	.....	24.339			
6	0.0608	0.0606	0.2285	0.0607	0.0541	8.6	38	1.4093	.....	31.836			
7	0.0733	0.0727	0.2978	0.0730	0.0706	7.5	39	1.4145	.....	31.908			
8	0.0814	0.0809	0.3470	0.0812	0.0822	6.6	40	1.4120	.....	31.971	1.4128	7.5775	35.5
9	0.0820	0.0820	0.3495	0.0820	0.0828	5.9	41	1.4131	.....	32.219			
10	0.0921	0.0920	0.4082	0.0920	0.0967	4.1	42	1.4152	.....	31.953			
11	0.1044	0.1038	0.4897	0.1041	0.1160	3.7	43	1.5991	.....	39.684			
12	0.1251	0.1239	0.6525	0.1245	0.1546	5.7	44	1.6032	.....	39.722			
13	0.1457	0.1454	0.8193	0.1456	0.1942	5.0	45	1.6025	.....	39.892	1.5991	9.4149	39.8
14	0.1671	0.1663	1.0018	0.1667	0.2374	4.7	46	1.5985	.....	39.840			
15	0.1876	0.1864	1.1933	0.1870	0.2828	5.0	47	1.5920	.....	39.515			
16	0.2078	0.2066	1.3970	0.2072	0.3310	5.4	48	1.7933	.....	48.966			
17	0.2539	0.2524	1.8940	0.2532	0.4488	5.8	49	1.7978	.....	49.220			
18	0.3031	0.3019	2.4868	0.3025	0.5893	6.4	50	1.7980	.....	49.354	1.7970	11.649	45.2
19	0.3494	0.3478	3.1138	0.3486	0.7379	7.7	51	1.7980	.....	49.245			
20	0.4021	0.4014	3.8803	0.4018	0.9195	7.8	52	1.7980	.....	49.012			
21	0.4517	0.4515	4.6650	0.4516	1.1055	9.4	53	1.9955	.....	60.813			
22	0.8000	0.8031	11.875				54	2.0026	.....	60.223			
23	0.7994	0.8043	11.894	0.8016	2.8192	18.0	55	2.0016	.....	61.057	2.0001	14.434	53.2
24	0.8000	0.8026	11.900				56	2.0037	.....	61.383			
25	0.8986	.....	14.622				57	1.9972	.....	61.078			
26	0.8971	.....	14.624	0.8979	3.4635	22.2							
27	0.8979	.....	14.603										
28	1.0043	.....	17.665										
29	1.0043	.....	17.676										
30	1.0039	.....	17.661										
31	1.0056	.....	17.663	1.0044	4.1858	24.9							
32	1.0038	.....	17.654										

TABLE 14.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES F.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0289	0.0285	0.0752	0.0287	0.0178	9.9
2	0.0337	0.0342	0.1008	0.0340	0.0239	14.4
3	0.0354	0.0351	0.1063	0.0352	0.0252	14.5
4	0.0424	0.0421	0.1327	0.0422	0.0314	8.7
5	0.0490	0.0484	0.1632	0.0487	0.0387	8.1
6	0.0571	0.0567	0.2038	0.0569	0.0483	6.9
7	0.0638	0.0636	0.2415	0.0637	0.0572	6.9
8	0.0738	0.0735	0.2978	0.0736	0.0706	6.2
9	0.0848	0.0842	0.3628	0.0845	0.0860	5.1
10	0.0949	0.0947	0.4295	0.0948	0.1018	4.7
11	0.1069	0.1067	0.5057	0.1068	0.1198	3.0
12	0.1206	0.1205	0.6000	0.1206	0.1422	1.9
13	0.1416	0.1412	0.7683	0.1414	0.1821	2.8
14	0.1598	0.1595	0.9137	0.1596	0.2165	2.0
15	0.1802	0.1796	1.0943	0.1799	0.2593	2.0
16	0.2001	0.1996	1.2803	0.1998	0.3034	2.0
17	0.2509	0.2503	1.8062	0.2506	0.4280	2.5
18	0.2996	0.2978	2.3397	0.2987	0.5544	2.0
19	0.3529	0.3522	3.0310	0.3526	0.7182	3.0
20	0.4000	0.3997	3.6570	0.3998	0.8666	2.9
21	0.4508	0.4507	4.4080	0.4508	1.0446	3.6
22	0.5012	0.4997	5.2037	0.5008	1.2299	4.2
23	0.5014	0.5009	5.1763			
24	0.6991	0.7018	8.8017	0.7010	2.0858	6.7
25	0.7012	0.7018	8.8025			
26	0.7990	0.7994	10.840	0.7990	2.5675	8.0
27	0.7983	0.7994	10.824			
28	0.7986	0.7992	10.841			
29	0.9028	0.9028	13.125	0.9027	3.1182	9.2
30	0.9043	0.9026	13.175			
31	0.9019	0.9021	13.178			
32	0.9996	.....	15.470	1.0028	3.6964	10.5
33	1.0033	.....	15.628			
34	1.0036	.....	15.625			
35	1.0043	.....	15.635			
36	1.0030	.....	15.638			
37	1.0022	.....	15.910	1.0017	3.7803 (Fence 2)	13.2
38	1.0029	.....	16.002			
39	1.0014	.....	15.951			
40	1.0012	.....	15.968			
41	1.0009	.....	15.933	1.0032	3.6964 (Fence 4)	10.5
42	1.0041	.....	15.559			
43	1.0018	.....	15.525			
44	1.0044	.....	15.665			
45	1.0014	.....	15.618			
46	1.0044	.....	15.628			
47	1.2036	.....	21.014	1.2035	4.9718	13.1
48	1.2043	.....	21.005			
49	1.2023	.....	20.959			
50	1.2030	.....	20.927			
51	1.2043	.....	20.998			
52	1.3988	.....	26.832	1.3977	6.3701	15.8
53	1.3994	.....	26.984			
54	1.3974	.....	26.891			
55	1.3995	.....	26.888			
56	1.3983	.....	26.814			
57	1.6008	.....	33.751	1.6012	7.9761	18.2
58	1.6016	.....	33.644			
59	1.6034	.....	33.692			
60	1.5969	.....	33.578			
61	1.6034	.....	33.630			
62	1.8050	.....	41.271	1.8021	9.8010	21.7
63	1.8028	.....	41.307			
64	1.8055	.....	41.334			
65	1.7991	.....	41.494			
66	1.7982	.....	41.392			
67	1.9992	.....	48.962	2.0008	11.659	23.7
68	2.0045	.....	49.561			
69	1.9953	.....	49.069			
70	2.0065	.....	49.432			
71	1.9957	.....	48.980			
72	2.0018	.....	51.410	2.0024	12.219 (Fence 1)	29.5
73	2.0009	.....	51.468			
74	2.0098	.....	51.403			
75	1.9944	.....	51.886			
76	2.0050	.....	51.652			
77	1.9975	.....	50.597	1.9928	11.931 (Fence 3)	27.4
78	1.9869	.....	50.070			
79	1.9973	.....	50.389			
80	1.9923	.....	50.211			
81	1.9902	.....	50.473			
82	2.1990	.....	56.717	2.2076	13.685	25.3
83	2.2049	.....	57.860			
84	2.2156	.....	57.986			
85	2.2062	.....	58.301			
86	2.2124	.....	57.895			
87	2.3622	.....	66.467	2.3638	15.891	31.3
88	2.3665	.....	67.729			
89	2.3656	.....	66.939			
90	2.3637	.....	68.418			
91	2.3610	.....	65.757			

TABLE 15.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES G.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0176	0.0182	0.0802	0.0179	0.0072	— 8.9
2	0.0306	0.0304	0.0860	0.0305	0.0204	+14.6
3	0.0319	0.0321	0.0972	0.0320	0.0230	20.4
4	0.0356	0.0352	0.1062	0.0354	0.0252	13.5
5	0.0415	0.0412	0.1318	0.0414	0.0312	11.4
6	0.0477	0.0464	0.1588	0.0470	0.0376	10.9
7	0.0537	0.0532	0.1890	0.0534	0.0448	9.0
8	0.0595	0.0588	0.2190	0.0592	0.0519	8.4
9	0.0676	0.0663	0.2607	0.0670	0.0618	6.9
10	0.0778	0.0763	0.3180	0.0770	0.0754	5.9
11	0.0883	0.0869	0.3825	0.0876	0.0906	5.0
12	0.0997	0.0992	0.4587	0.0994	0.1087	4.2
13	0.1215	0.1206	0.6102	0.1210	0.1446	3.1
14	0.1420	0.1408	0.7663	0.1414	0.1816	2.5
15	0.1646	0.1643	0.9568	0.1644	0.2267	2.1
16	0.1820	0.1814	1.1088	0.1817	0.2627	1.9
17	0.1991	0.1984	1.2655	0.1988	0.2999	1.6
18	0.2495	0.2483	1.7733	0.2489	0.4202	1.6
19	0.3002	0.2992	2.3373	0.2997	0.5539	1.4
20	0.3499	0.3497	2.9572	0.3498	0.7008	1.7
21	0.4014	0.4014	3.6262	0.4014	0.8593	1.5
22	0.4511	0.4500	4.3160	0.4506	1.0227	1.5
23	0.5004	0.4996	5.0917	0.5000	1.2065	2.5
24	0.5003	0.4996	5.0910			
25	0.8025	0.8032	10.529	0.8028	2.4957	4.2
26	0.8029	0.8024	10.554			
27	0.8023	0.8032	10.513			
28	0.9032	0.9049	12.667	0.9041	3.0087	5.1
29	0.9028	0.9050	12.700			
30	0.9036	0.9051	12.723			
31	1.0022	.....	14.955	1.0013	3.5432	6.2
32	1.0019	.....	14.955			
33	1.0006	.....	14.949			
34	1.0015	.....	14.947			
35	1.0005	.....	14.953			
36	0.9976	.....	15.699	0.9975	3.7209 (Fence 1)	12.2
37	0.9977	.....	15.684			
38	0.9969	.....	15.705			
39	0.9992	.....	15.723			
40	0.9960	.....	15.700			
41	1.0036	.....	14.959	1.0081	3.5512 (Fence 3)	6.1
42	1.0036	.....	15.014			
43	1.0036	.....	14.967			
44	1.0026	.....	14.983			
45	1.0021	.....	15.008			
46	1.2112	.....	20.196	1.2154	4.8137	7.9
47	1.2152	.....	20.306			
48	1.2169	.....	20.377			
49	1.2187	.....	20.415			
50	1.2150	.....	20.276			
51	1.4056	.....	25.671	1.4043	6.0822	9.8
52	1.4037	.....	25.712			
53	1.4031	.....	25.688			
54	1.4042	.....	25.616			
55	1.4051	.....	25.697			
56	1.6045	.....	31.941	1.6016	7.5399	11.7
57	1.6021	.....	31.768			
58	1.6007	.....	31.789			
59	1.6055	.....	31.953			
60	1.5954	.....	31.640			
61	1.8076	.....	38.884	1.8081	9.2130	13.8
62	1.8084	.....	38.766			
63	1.8075	.....	38.879			
64	1.8070	.....	38.862			
65	1.8098	.....	39.003			
66	1.9911	.....	45.920	1.9936	10.883	16.1
67	1.9891	.....	46.011			
68	1.9974	.....	45.981			
69	1.9971	.....	45.871			
70	1.9932	.....	45.857			
71	2.0011	.....	50.194	1.9944	11.797 (Fence 2)	25.8
72	2.0044	.....	49.858			
73	2.0016	.....	49.925			
74	1.9787	.....	49.744			
75	1.9864	.....	49.188			
76	1.9899	.....	46.518	1.9882	11.008 (Fence 4)	17.9
77	1.9918	.....	46.625			
78	1.9850	.....	46.270			
79	1.9848	.....	46.413			
80	1.9897	.....	46.442			
81	2.1881	.....	53.947	2.1794	12.682	18.4
82	2.1883	.....	53.747			
83	2.1856	.....	53.643			
84	2.1792	.....	53.591			
85	2.1537	.....	52.671			
86	2.4101	.....	63.476	2.4112	15.073	20.9
87	2.4069	.....	63.658			
88	2.4110	.....	63.360			
89	2.4108	.....	63.614			
90	2.4174	.....	63.937			

TABLE 16.—Schoder and Turner, Sharp-Crested Weirs, Series H.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0222	0.0218	0.0478	0.0220	0.0113	3.7
2	0.0342	0.0337	0.1127	0.0340	0.0267	27.8
3	0.0459	0.0451	0.1467	0.0455	0.0348	7.4
4	0.0547	0.0543	0.1915	0.0545	0.0454	7.1
5	0.0611	0.0603	0.2233	0.0607	0.0529	6.2
6	0.0704	0.0694	0.2765	0.0699	0.0655	6.3
7	0.0797	0.0789	0.3283	0.0793	0.0778	4.7
8	0.0892	0.0882	0.3852	0.0887	0.0913	3.8
9	0.1005	0.1002	0.4607	0.1004	0.1092	3.1
10	0.1212	0.1205	0.6033	0.1208	0.1430	2.3
11	0.1402	0.1388	0.7445	0.1395	0.1764	1.7
12	0.1601	0.1591	0.9098	0.1596	0.2156	1.6
13	0.1806	0.1802	1.0867	0.1804	0.2575	0.9
14	0.2002	0.1996	1.2627	0.1999	0.2992	0.5
15	0.2488	0.2472	1.7472	0.2480	0.4140	0.7
16	0.3013	0.3001	2.3353	0.3007	0.5534	0.8
17	0.3506	0.3500	2.9402	0.3503	0 6967	0.9
18	0.4016	0.4007	3.6071	0.4012	0.8548	1.0
19	0.4512	0.4506	4.2986	0.4509	1.0186	1.0
20	0.5025	0.5020	5.0533	0.5024	1.1977	1.0
21	0.5030	0.5021	5.0557			
22	0.8014	0.8026	10.316			
23	0.8019	0.8019	10.332	0.8019	2.4473	2.3
24	0.8018	0.8020	10.335			
25	0.9007	0.9012	12.384			
26	0.9006	0.9019	12.393	0.9010	2.9368	3.1
27	0.9010	0.9008	12.403			
28	1.0024	.....	14.611			
29	1.0022	.....	14.626			
30	1.0027	.....	14.638	1.0025	3.4671	3.7
31	1.0025	.....	14.649			
32	1.0027	.....	14.631			
33	1.0037	.....	14.522			
34	1.0030	.....	14.493			
35	1.0036	.....	14.511	1.0034	3.4429 (Fence 1)	2.9
36	1.0041	.....	14.569			
37	1.0028	.....	14.550			
38	1.0034	.....	15.005			
39	1.0030	.....	15.008			
40	1.0033	.....	14.944	1.0030	3.5500 (Fence 4)	6.1
41	1.0023	.....	15.005			
42	1.0032	.....	14.944			
43	1.2015	.....	19.356			
44	1.2014	.....	19.352			
45	1.2026	.....	19.351	1.2012	4.5886	4.7
46	1.2005	.....	19.395			
47	1.2000	.....	19.365			
48	1.4005	.....	24.703			
49	1.3960	.....	24.553			
50	1.3988	.....	24.637	1.3982	5.8423	6.1
51	1.3992	.....	24.706			
52	1.3965	.....	24.674			
53	1.6148	.....	31.125			
54	1.6046	.....	30.837			
55	1.6159	.....	31.198	1.6127	7.3632	8.0
56	1.6090	.....	30.925			
57	1.6190	.....	31.279			
58	1.8113	.....	37.723			
59	1.8078	.....	37.453			
60	1.8083	.....	37.369	1.8078	8.8850	9.8
61	1.8094	.....	37.528			
62	1.8022	.....	37.401			
63	2.0036	.....	44.224			
64	2.0098	.....	44.445			
65	2.0126	.....	44.758	2.0085	10.550	11.3
66	2.0159	.....	44.815			
67	2.0004	.....	44.357			
68	1.9994	.....	43.148			
69	1.9984	.....	43.176			
70	1.9961	.....	43.175	1.9974	10.215 (Fence 2)	8.7
71	1.9977	.....	42.959			
72	1.9953	.....	43.086			
73	1.9955	.....	49.884			
74	1.9974	.....	49.819			
75	2.0040	.....	49.874	1.9987	11.825 (Fence 3)	25.7
76	2.0000	.....	50.142			
77	1.9965	.....	49.786			
78	2.2052	.....	52.961			
79	2.1987	.....	52.540			
80	2.2053	.....	52.748	2.2043	12.506	14.8
81	2.2033	.....	52.771			
82	2.2088	.....	52.867			
83	2.4680	.....	63.172			
84	2.4716	.....	63.259			
85	2.4758	.....	63.234	2.4746	15.009	15.8
86	2.4791	.....	63.442			
87	2.4787	.....	63.576			

TABLE 17.—Schoder and Turner, Sharp-Crested Weirs, Series I.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0224	0.0229	0.0503	0.0226	0.0119	5.3
2	0.0266	0.0265	0.0657	0.0266	0.0156	7.6
3	0.0349	0.0355	0.1058	0.0352	0.0251	14.1
4	0.0422	0.0425	0.1340	0.0424	0.0318	9.3
5	0.0518	0.0525	0.1808	0.0522	0.0428	7.8
6	0.0602	0.0605	0.2225	0.0604	0.0527	6.7
7	0.0712	0.0714	0.2838	0.0713	0.0673	6.2
8	0.0815	0.0817	0.3412	0.0816	0.0816	5.2
9	0.0902	0.0905	0.3987	0.0904	0.0945	4.4
10	0.1004	0.1004	0.4645	0.1004	0.1101	4.0
11	0.1167	0.1166	0.5752	0.1166	0.1363	2.8
12	0.1404	0.1406	0.7535	0.1405	0.1786	1.8
13	0.1624	0.1626	0.9366	0.1625	0.2219	1.7
14	0.1836	0.1844	1.1192	0.1840	0.2652	0.9
15	0.2010	0.2006	1.2778	0.2008	0.3028	1.1
16	0.2487	0.2485	1.7458	0.2486	0.4137	0.2
17	0.3023	0.3018	2.3483	0.3020	0.5565	0.7
18	0.3517	0.3512	2.9412	0.3514	0.6970	0.5
19	0.4070	0.4070	3.6612	0.4070	0.8676	0.3
20	0.4542	0.4543	4.3078	0.4542	1.0208	0.1
21	0.5048	0.5054	5.0390	0.5050	1.1927	— 0.2
22	0.5045	0.5052	5.0270			
23	0.8012	0.8016	10.121	0.8010	2.3970	0.4
24	0.8009	0.8014	10.132			
25	0.8004	0.8006	10.093			
26	0.9026	0.9031	12.203	0.9040	2.8957	1.2
27	0.9031	0.9046	12.196			
28	0.9048	0.9058	12.261			
29	1.0015	.....	14.311	0.9995	3.383	1.6
30	0.9979	.....	14.228			
31	0.9984	.....	14.254			
32	0.9990	.....	14.266			
33	1.0008	.....	14.286			
34	1.2021	.....	18.924	1.2023	4.4934	2.4
35	1.2024	.....	18.970			
36	1.2024	.....	18.973			
37	1.2031	.....	18.986			
38	1.2017	.....	18.957			
39	1.3981	.....	23.929	1.3967	5.6715	3.2
40	1.3974	.....	23.973			
41	1.3963	.....	23.965			
42	1.3952	.....	23.914			
43	1.3967	.....	23.888			
44	1.5961	.....	29.603	1.6028	7.0475	4.3
45	1.6019	.....	29.683			
46	1.6072	.....	29.861			
47	1.6045	.....	29.824			
48	1.6042	.....	29.732			
49	1.8083	.....	35.909	1.8097	8.5536	5.5
50	1.8075	.....	36.105			
51	1.8116	.....	36.201			
52	1.8106	.....	36.096			
53	1.8107	.....	36.170			
54	1.9991	.....	42.639	2.0009	10.088	7.0
55	1.9967	.....	42.503			
56	2.0004	.....	42.606			
57	2.0042	.....	42.579			
58	2.0042	.....	42.548			
59	2.0147	.....	41.495	2.0029	9.7786 (Fence 1)	3.6
60	2.0025	.....	41.145			
61	2.0005	.....	41.331			
62	1.9985	.....	41.212			
63	1.9983	.....	41.146			
64	2.0098	.....	41.890	2.0057	9.8945 (Fence 2)	4.6
65	2.0055	.....	41.756			
66	2.0059	.....	41.796			
67	2.0028	.....	41.621			
68	2.0043	.....	41.710			
69	2.0086	.....	46.791	2.0039	11.094 (Fence 3)	17.5
70	2.0019	.....	46.687			
71	2.0048	.....	46.907			
72	2.0013	.....	46.825			
73	2.0029	.....	46.872			
74	2.0036	.....	44.375	2.0039	10.516 (Fence 4)	11.3
75	2.0036	.....	44.400			
76	2.0052	.....	44.284			
77	2.0045	.....	44.420			
78	2.0025	.....	44.410			
79	2.2094	.....	49.752	2.2018	11.757	8.1
80	2.1972	.....	49.642			
81	2.2020	.....	49.653			
82	2.1982	.....	49.478			
83	2.2023	.....	49.544			
84	2.4129	.....	57.647	2.4140	13.621	9.1
85	2.4118	.....	57.461			
86	2.4152	.....	57.596			
87	2.4165	.....	57.379			
88	2.4136	.....	57.318			
89	2.6661	.....	67.792	2.6691	16.092	10.8
90	2.6656	.....	67.631			
91	2.6682	.....	67.974			
92	2.6678	.....	67.680			
93	2.6776	.....	68.462			

TABLE 18.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES J.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float gauge.	Plumb-bob gauge.				
1	0.0171	0.0169	0.0247	0.0170	0.0059	—20.2
2	0.0260	0.0261	0.0625	0.0260	0.0148	+ 5.7
3	0.0355	0.0360	0.1083	0.0358	0.0257	14.2
4	0.0445	0.0443	0.1418	0.0444	0.0336	8.0
5	0.0521	0.0520	0.1785	0.0520	0.0423	7.1
6	0.0622	0.0620	0.2302	0.0621	0.0545	5.8
7	0.0696	0.0698	0.2717	0.0697	0.0644	5.1
8	0.0805	0.0809	0.3377	0.0807	0.0800	4.7
9	0.0888	0.0885	0.3848	0.0886	0.0912	3.9
10	0.1032	0.1034	0.4880	0.1033	0.1145	3.5
11	0.1211	0.1211	0.6068	0.1211	0.1438	2.4
12	0.1393	0.1398	0.7532	0.1396	0.1785	2.8
13	0.1622	0.1618	0.9280	0.1620	0.2199	1.3
14	0.1788	0.1788	1.0703	0.1788	0.2536	0.7
15	0.2026	0.2030	1.2987	0.2028	0.3077	1.2
16	0.2526	0.2526	1.7938	0.2526	0.4251	0.5
17	0.3022	0.3017	2.3372	0.3020	0.5538	0.2
18	0.3507	0.3507	2.9225	0.3507	0.6925	+ 0.1
19	0.4043	0.4052	3.6110	0.4048	0.8557	— 0.2
20	0.4529	0.4534	4.2775	0.4532	1.0136	— 0.4
21	0.5045	0.5052	5.0333	0.5049	1.1899	— 0.4
22	0.5049	0.5051	5.0093			
23	0.8001	0.8004	10.014			
24	0.8002	0.7999	10.038	0.8000	2.3778	— 0.2
25	0.7998	0.7998	10.051			
26	0.9019	0.9020	12.037			
27	0.9016	0.9030	12.104	0.9020	2.8626	+ 0.3
28	0.9013	0.9022	12.099			
29	0.9990	.....	14.151			
30	0.9991	.....	14.161			
31	0.9994	.....	14.170	0.9993	3.3559	0.9
32	0.9991	.....	14.162			
33	1.0001	.....	14.165			
34	1.2021	.....	18.744			
35	1.2021	.....	18.795			
36	1.2010	.....	18.797	1.2019	4.4539	1.5
37	1.2018	.....	18.811			
38	1.2025	.....	18.831			
39	1.4004	.....	23.863			
40	1.3993	.....	23.821			
41	1 3986	.....	23.798	1.3991	5.6458	2.4
42	1.4003	.....	23.848			
43	1.3971	.....	23.796			
44	1.6020	.....	29.458			
45	1.6056	.....	29.501			
46	1.6051	.....	29.516	1.6052	6.9918	3.2
47	1.6071	.....	29.539			
48	1.6062	.....	29.512			
49	1.8006	.....	35.245			
50	1.7984	.....	35.435			
51	1.7976	.....	35.259	1.7981	8.3683	4.2
52	1.7992	.....	35.379			
53	1.7945	.....	35.253			
54	2.0015	.....	41.772			
55	1.9972	.....	41.799			
56	2.0016	.....	41.948	2.0012	9.9319	5.4
57	2.0026	.....	41.969			
58	2.0030	.....	42.075			
59	1.9983	.....	40.439			
60	2.0008	.....	40.395			
61	1.9966	.....	40.410	1.9997	9.5863 (Fence 1)	1.8
62	2.0008	.....	40.479			
63	2.0020	.....	40.549			
64	1.9918	.....	40.101			
65	1.9941	.....	40.056			
66	1.9945	.....	40.037	1.9950	9.4939 (Fence 2)	1.2
67	1.9981	.....	40.081			
68	1.9963	.....	40.046			
69	1.9941	.....	44.042			
70	1.9986	.....	44.370			
71	1.9983	.....	44.416	1.9946	10.521 (Fence 3)	1.5
72	1.9990	.....	44.691			
73	1.9830	.....	44.480			
74	1.9939	.....	42.822			
75	2.0025	.....	42.925			
76	2.0070	.....	43.212	2.0036	10.198 (Fence 4)	8.0
77	2.0080	.....	43.068			
78	2.0064	.....	43.158			
79	2.2033	.....	48.923			
80	2.2017	.....	48.777			
81	2.2042	.....	48.728	2.2028	11.564	6.2
82	2.2036	.....	48.715			
83	2.2014	.....	48.854			
84	2.4061	.....	56.462			
85	2.4025	.....	56.240			
86	2.4025	.....	56.238	2.4043	13.341	7.5
87	2.3990	.....	56.347			
88	2.4112	.....	56.215			
89	2.7001	.....	67.789			
90	2.6924	.....	67.587			
91	2.7070	.....	68.124	2.6988	16.040	8.6
92	2.7033	.....	67 592			
93	2.6911	.....	67.361			

TABLE 19.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES K.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0136	0.0137	0.0125	0.0136	0.0030	−43.4
2	0.0190	0.0196	0.0369	0.0193	0.0087	− 2.2
3	0.0279	0.0285	0.0730	0.0282	0.0173	+ 9.5
4	0.0368	0.0372	0.1143	0.0370	0.0271	14.4
5	0.0461	0.0465	0.1560	0.0463	0.0370	11.4
6	0.0539	0.0545	0.1918	0.0542	0.0455	8.3
7	0.0630	0.0636	0.2410	0.0633	0.0571	7.5
8	0.0709	0.0710	0.2815	0.0710	0.0667	5.9
9	0.0815	0.0821	0.3482	0.0818	0.0825	5.9
10	0.0930	0.0933	0.4197	0.0932	0.0995	5.1
11	0.0981	0.0985	0.4515	0.0983	0.1070	4.3
12	0.1172	0.1174	0.5822	0.1173	0.1380	3.1
13	0.1390	0.1395	0.7467	0.1392	0.1769	2.3
14	0.1639	0.1644	0.9508	0.1642	0.2253	1.7
15	0.1799	0.1804	1.0873	0.1806	0.2587	1.2
16	0.1812	0.1809	1.0962			
17	0.2001	0.2002	1.2717	0.2002	0.3014	1.0
18	0.2515	0.2509	1.7850	0.2512	0.4230	0.9
19	0.3014	0.3012	2.3345	0.3013	0.5532	0.5
20	0.3508	0.3507	2.9275	0.3508	0.6937	0.3
21	0.3999	0.4001	3.5583	0.4000	0.8432	0.1
22	0.4509	0.4504	4.2494	0.4506	1.0070	0.0
23	0.4996	0.5000	4.9653	0.4998	1.1762	0.0
24	0.4999	0.4997	4.9617			
25	0.8023	.....	10.105			
26	0.8021	.....	10.080	0.8022	2.3914	− 0.1
27	0.8021	.....	10.090			
28	0.8987	.....	11.987			
29	0.8988	.....	11.990	0.8987	2.8413	+ 0.1
30	0.8985	.....	11.994			
31	0.9989	.....	14.023			
32	0.9994	.....	14.078			
33	0.9990	.....	14.077	0.9989	3.3354	0.3
34	0.9991	.....	14.110			
35	0.9983	.....	14.090			
36	1.2072	.....	18.571			
37	1.2068	.....	18.807			
38	1.2072	.....	18.813	1.2071	4.4440	0.6
39	1.2076	.....	18.809			
40	1.2065	.....	18.768			
41	1.4007	.....	23.619			
42	1.3998	.....	23.681			
43	1.3968	.....	23.595	1.3989	5.5988	1.6
44	1.3994	.....	23.629			
45	1.3976	.....	23.611			
46	1.6011	.....	29.302			
47	1.6031	.....	29.301			
48	1.5996	.....	29.294	1.6017	6.9408	2.8
49	1.6027	.....	29.305			
50	1.6021	.....	29.248			
51	1.8067	.....	34.995	1.8057	8.3300	3.1
52	1.8055	.....	35.186			
53	1.8053	.....	35.195			
54	1.8044	.....	35.175			
55	1.8066	.....	35.212			
56	2.0115	.....	41.597	2.0062	9.8371	4.0
57	2.0116	.....	41.611			
58	2.0150	.....	41.663			
59	2.0032	.....	41.259			
60	2.0099	.....	41.432			
61	2.0072	.....	40.110			
62	2.0109	.....	40.209			
63	2.0095	.....	40.223	2.0067	9.5078 (Fence 1)	+ 0.4
64	2.0029	.....	40.100			
65	2.0032	.....	39.972			
66	2.0034	.....	39.709			
67	2.0000	.....	39.584			
68	2.0016	.....	39.504	1.9987	9.3869 (Fence 2)	− 0.2
69	1.9934	.....	39.646			
70	1.9951	.....	39.620			
71	1.9977	.....	43.826			
72	2.0021	.....	44.192			
73	1.9981	.....	44.008	1.9992	10.436 (Fence 3)	+10.9
74	2.0005	.....	44.232			
75	1.9974	.....	43.938			
76	1.9951	.....	43.275			
77	1.9965	.....	43.253			
78	1.9953	.....	43.245	1.9956	10.251 (Fence 4)	9.2
79	1.9948	.....	43.278			
80	1.9961	.....	43.244			
81	2.0001	.....	42.058			
82	1.9975	.....	42.379			
83	1.9772	.....	41.892	1.9927	10.005 (Fence 5)	6.8
84	1.9965	.....	42.466			
85	1.9921	.....	42.320			
86	2.2047	.....	48.494			
87	2.1970	.....	48.218			
88	2.2035	.....	48.335	2.2028	11.466	5.3
89	2.2044	.....	48.460			
90	2.2044	.....	48.435			
91	2.4162	.....	56.360			
92	2.4090	.....	56.121			
93	2.4068	.....	55.932	2.4108	13.295	6.7
94	2.4095	.....	56.024			
95	2.4127	.....	56.085			
96	2.6524	.....	65.743			
97	2.6489	.....	64.825			
98	2.6579	.....	65.281	2.6535	15.444	7.3
99	2.6555	.....	64.934			
100	2.6526	.....	65.085			

TABLE 20.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES L.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0185	0.0187	0.0360	0.0186	0.0085	1.2
2	0.0277	0.0274	0.0747	0.0276	0.0177	15.7
3	0.0340	0.0336	0.1060	0.0338	0.0251	21.3
4	0.0453	0.0455	0.1520	0.0454	0.0360	11.8
5	0.0545	0.0543	0.1942	0.0544	0.0460	8.7
6	0.0622	0.0618	0.2378	0.0620	0.0564	9.7
7	0.0735	0.0731	0.2992	0.0733	0.0709	7.3
8	0.0829	0.0826	0.3552	0.0828	0.0842	6.2
9	0.0909	0.0906	0.4062	0.0908	0.0963	5.7
10	0.1005	0.1000	0.4658	0.1002	0.1104	4.5
11	0.1195	0.1194	0.6027	0.1194	0.1428	3.9
12	0.1410	0.1406	0.7637	0.1408	0.1810	2.9
13	0.1622	0.1622	0.9433	0.1622	0.2235	2.8
14	0.1832	0.1837	1.1252	0.1834	0.2666	2.0
15	0.2009	0.2004	1.2850	0.2006	0.3045	1.8
16	0.2528	0.2521	1.8030	0.2524	0.4273	1.2
17	0.2980	0.2974	2.3008	0.2977	0.5452	0.8
18	0.3514	0.3507	2.9452	0.3510	0.6979	0.8
19	0.4010	0.4007	3.5793	0.4008	0.8482	0.4
20	0.4534	0.4530	4.2958	0.4532	1.0180	0.2
21	0.5011	0.5000	4.9730	0.5005	1.1788	0.0
22	0.5008	0.4999	4.9760			
23	0.9018	.....	12.061			
24	0.9015	.....	12.066			
25	0.9024	.....	12.088	0.9019	2.8606	0.3
26	0.9021	.....	12.083			
27	0.9016	.....	12.061			
28	1.5970	.....	29.044			
29	1.5990	.....	29.008			
30	1.5965	.....	29.081	1.5968	6.8852	2.5
31	1.5986	.....	29.081			
32	1.5931	.....	29.065			
33	2.2508	.....	50.476			
34	2.2488	.....	50.427			
35	2.2387	.....	49.997	2.2451	11.914	6.4
36	2.2375	.....	50.075			
37	2.2497	.....	50.409			

TABLE 21.—Schoder and Turner, Sharp-Crested Weirs, Series M.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0147	0.0146	0.0188	0.0146	0.0045	−23.7	38	1.4057	.....	23.880			
2	0.0297	0.0296	0.0825	0.0296	0.0195	+15.4	39	1.4068	.....	23.959			
3	0.0347	0.0347	0.1028	0.0347	0.0244	13.5	40	1.4079	.....	23.961	1.4065	5.6731	2.1
4	0.0458	0.0457	0.1470	0.0458	0.0348	6.4	41	1.4067	.....	23.973			
5	0.0588	0.0589	0.2125	0.0588	0.0504	6.1	42	1.4056	.....	23.929			
6	0.0695	0.0690	0.2672	0.0692	0.0633	4.3	43	1.5938	.....	29.222			
7	0.0816	0.0814	0.3408	0.0815	0.0808	4.3	44	1.5956	.....	29.142			
8	0.0900	0.0898	0.3927	0.0896	0.0931	4.3	45	1.5952	.....	29.102	1.5948	6.8997	2.9
9	0.0981	0.0978	0.4487	0.0980	0.1063	4.0	46	1.5975	.....	29.115			
10	0.1190	0.1196	0.5998	0.1193	0.1421	3.6	47	1.5919	.....	29.003			
11	0.1374	0.1381	0.7390	0.1378	0.1751	2.8	48	1.8046	.....	35.142			
12	0.1569	0.1578	0.8982	0.1574	0.2128	2.3	49	1.7996	.....	34.967			
13	0.1787	0.1795	1.0808	0.1791	0.2561	1.5	50	1.8015	.....	35.111	1.8019	8.3098	3.2
14	0.1979	0.1985	1.2552	0.1982	0.2974	1.2	51	1.8019	.....	35.057			
15	0.2466	0.2476	1.7420	0.2471	0.4128	0.9	52	1.8021	.....	35.060			
16	0.2982	0.2989	2.3140	0.2986	0.5483	0.9	53	1.9994	.....	41.283			
17	0.3462	0.3473	2.8942	0.3468	0.6858	0.8	54	1.9940	.....	41.159			
18	0.3981	0.3990	3.5579	0.3986	0.8431	0.6	55	1.9937	.....	41.143	1.9954	9.7606	4.0
19	0.4468	0.4471	4.2256	0.4470	1.0013	0.6	56	1.9921	.....	41.148			
20	0.4993	0.5003	4.9817	0.5000	1.1815	0.4	57	1.9978	.....	41.215			
21	0.4999	0.5006	4.9903				58	2.2131	.....	48.293			
22	0.8029	0.8042	10.118				59	2.2173	.....	48.299			
23	0.8025	0.8041	10.145	0.8034	2.4024	0.2	60	2.2142	.....	48.345	2.2136	11.456	4.5
24	0.8028	0.8038	10.151				61	2.2148	.....	48.333			
25	0.8990	0.8991	12.045				62	2.2086	.....	48.457			
26	0.8987	0.8992	12.041	0.8992	2.8536	0.5	63	2.4060	.....	55.352			
27	0.8998	0.8994	12.040				64	2.4141	.....	55.477			
28	0.9989	.....	14.093				65	2.4039	.....	55.300	2.4084	13.125	5.4
29	0.9975	.....	14.138				66	2.4061	.....	55.435			
30	0.9980	.....	14.128	0.9981	3.3460	0.8	67	2.4117	.....	55.383			
31	0.9978	.....	14.121				68	2.5201	.....	59.660			
32	0.9985	.....	14.121				69	2.5311	.....	59.675			
33	1.1995	.....	18.727				70	2.5284	.....	59.437	2.5272	14.136	5.7
34	1.1992	.....	18.722				71	2.5308	.....	59.624			
35	1.1974	.....	18.678	1.1983	4.4334	1.5	72	2.5256	.....	59.864			
36	1.1971	.....	18.703										
37	1.1984	.....	18.715										

TABLE 22.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES N.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0120	0.0126	0.0055	0.0123	0.0013	−71.8	51	0.9975	.....	16.414			
2	0.0217	0.0220	0.0458	0.0218	0.0109	+ 1.9	52	0.9993	.....	16.491			
3	0.0293	0.0296	0.0797	0.0294	0.0189	12.5	53	1.0002	.....	16.547	0.9986	3.9071	17.6
4	0.0342	0.0343	0.0995	0.0342	0.0236	11.8	54	0.9994	.....	16.509		(Fence 2)	
5	0.0422	0.0428	0.1298	0.0425	0.0308	5.5	55	0.9968	.....	16.481			
6	0.0416	0.0415	0.1262	0.0416	0.0299	5.7	56	1.2070	.....	21.711			
7	0.0423	0.0424	0.1297	0.0424	0.0307	5.5	57	1.2078	.....	21.808			
8	0.0512	0.0510	0.1707	0.0511	0.0405	5.2	58	1.2075	.....	21.782	1.2076	5.1609	16.8
9	0.0611	0.0611	0.2227	0.0611	0.0528	5.0	59	1.2084	.....	21.770			
10	0.0613	0.0609	0.2253	0.0611	0.0534	6.2	60	1.2078	.....	21.822			
11	0.0684	0.0680	0.2638	0.0682	0.0625	5.4	61	1.4118	.....	28.349			
12	0.0686	0.0683	0.2660	0.0684	0.0630	5.7	62	1.4051	.....	28.201			
13	0.0790	0.0788	0.3257	0.0789	0.0772	4.6	63	1.4038	.....	28.205	1.4078	6.7031	20.5
14	0.0902	0.0902	0.3937	0.0902	0.0933	3.4	64	1.4101	.....	28.358			
15	0.1016	0.1017	0.4705	0.1016	0.1115	3.3	65	1.4084	.....	28.320			
16	0.1217	0.1217	0.6137	0.1217	0.1454	2.8	66	1.6053	.....	35.067			
17	0.1412	0.1411	0.7645	0.1412	0.1812	2.5	67	1.6002	.....	35.108			
18	0.1591	0.1588	0.9153	0.1590	0.2169	2.7	68	1.6037	.....	35.254	1.6040	8.3491	23.4
19	0.1799	0.1800	1.1013	0.1800	0.2610	2.6	69	1.6064	.....	35.331			
20	0.2005	0.2004	1.2928	0.2004	0.3064	2.6	70	1.6043	.....	35.407			
21	0.2022	0.2023	1.3010	0.2022	0.3083	1.8	71	1.8672	.....	45.126			
22	0.2535	0.2536	1.8518	0.2536	0.4388	3.2	72	1.8578	.....	44.726			
23	0.2989	0.2986	2.3800	0.2988	0.5640	3.7	73	1.8492	.....	44.472	1.8493	10.537	25.8
24	0.3490	0.3488	3.0148	0.3489	0.7144	4.1	74	1.8396	.....	44.146			
25	0.4014	0.4015	3.7436	0.4014	0.8871	4.7	75	1.8328	.....	43.862			
26	0.4534	0.4536	4.5073				76	2.0094	.....	51.870			
27	0.4534	0.4541	4.5407	0.4536	1.0727	5.5	77	2.0122	.....	51.987			
28	0.4537	0.4530	4.5300				78	2.0042	.....	51.457	2.0087	12.266	29.4
29	0.4537	0.4536	4.5283				79	2.0126	.....	52.005			
30	0.5006	0.5009	5.2617	0.5006	1.2482	5.8	80	2.0051	.....	51.490			
31	0.5005	0.5006	5.2730				81	2.0040	.....	54.017			
32	0.7030	.....	9.0230	0.7033	2.1453	9.2	82	1.9908	.....	53.595			
33	0.7034	.....	9.0663				83	1.9998	.....	53.401	2.0009	12.692	34.7
34	0.7036	.....	9.0705				84	2.0032	.....	53.317		(Fence 1)	
35	0.8033	.....	11.234	0.8038	2.6647	11.0	85	2.0067	.....	53.480			
36	0.8042	.....	11.255				86	2.0020	.....	50.167			
37	0.8039	.....	11.246				87	2.0078	.....	50.460			
38	0.9031	.....	13.598	0.9029	3.2204	12.7	88	2.0088	.....	50.505	2.0068	11.950	26.2
39	0.9030	.....	13.589				89	2.0067	.....	50.343		(Fence 2)	
40	0.9026	.....	13.584				90	2.0092	.....	50.661			
41	1.0031	.....	16.122	1.0031	3.8228	+14.3	91	2.1968	.....	60.354			
42	1.0024	.....	16.143				92	2.2064	.....	60.740			
43	1.0033	.....	16.118				93	2.2014	.....	60.588	2.2012	14.316	31.6
44	1.0036	.....	16.142				94	2.2011	.....	60.405			
45	1.0030	.....	16.137				95	2.2002	.....	59.979			
46	1.0084	.....	16.458	1.0069	3.9047 (Fence 1)	16.1	96	2.3336	.....	67.666			
47	1.0074	.....	16.492				97	2.3341	.....	67.987			
48	1.0029	.....	16.466				98	2.3368	.....	67.925	2.3301	16.062	35.6
49	1.0080	.....	16.484				99	2.3293	.....	67.604			
50	1.0080	.....	16.489				100	2.3166	.....	67.724			

TABLE 23.—Schoder and Turner, Sharp-Crested Weirs, Series O.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet. per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0511	0.0515	0.1700	0.0513	0.0403	+4.1	40	1.6062	.....	29.348			
2	0.0509	0.0509	0.1685	0.0509	0.0399	4.2	41	1.6003	.....	29.301			
3	0.1013	0.1016	0.4650	0.1014	0.1102	2.5	42	1.6001	.....	29.254	1.6024	6.9478	+2.9
4	0.1016	0.1016	0.4585	0.1016	0.1086	0.6	43	1.6026	.....	29.396			
5	0.1219	0.1219	0.6102	0.1219	0.1446	2.0	44	1.6028	.....	29.319			
6	0.1406	0.1411	0.7497	0.1408	0.1777	0.9	45	1.8138	.....	35.734			
7	0.1404	0.1401	0.7417	0.1402	0.1758	0.6	46	1.8128	.....	35.685			
8	0.1612	0.1603	0.9123	0.1608	0.2162	0.7	47	1.8085	.....	35.483	1.8130	8.4562	4.0
9	0.1802	0.1799	1.0817	0.1800	0.2563	0.8	48	1.8136	.....	35.716			
10	0.2013	0.2016	1.2720	0.2014	0.3014	0.1	49	1.8163	.....	35.807			
11	0.2012	0.2009	1.2703	0.2010	0.3010	0.3	50	2.0037	.....	41.841			
12	0.2508	0.2501	1.7705	0.2504	0.4196	0.6	51	2.0055	.....	42.044			
13	0.2988	0.2981	2.3032	0.2984	0.5458	+0.6	52	2.0065	.....	41.969	2.0040	9.9255	5.1
14	0.3510	0.3502	2.9333	0.3506	0.6951	+0.5	53	2.0040	.....	42.015			
15	0.4002	0.4000	3.5560	0.4001	0.8427	0.0	54	2.0003	.....	41.559			
16	0.4530	0.4528	4.2761	0.4529	1.0133	−0.2	55	2.1961	.....	48.597			
17	0.5015	0.5012	4.9960	0.5013	1.1846	+0.2	56	2.1937	.....	48.357			
18	0.5015	0.5011	5.0023				57	2.1964	.....	48.443	2.1948	11.480	6.0
19	0.8007	.....	10.073				58	2.1940	.....	48.379			
20	0.8008	.....	10.102	0.8009	2.3911	+0.2	59	2.1937	.....	48.460			
21	0.8011	.....	10.096				60	2.3906	.....	55.539			
22	0.9014	.....	12.114				61	2.3978	.....	55.891			
23	0.9018	.....	12.110	0.9019	2.8705	+0.6	62	2.4006	.....	56.215	2.4040	13.298	7.1
24	0.9024	.....	12.117				63	2.4138	.....	56.419			
25	1.0018	.....	14.184				64	2.4170	.....	56.534			
26	1.0014	.....	14.193				65	2.5736	.....	62.996			
27	1.0019	.....	14.189	1.0016	3.3618	+0.7	66	2.5690	.....	62.519			
28	1.0020	.....	14.190				67	2.5570	.....	62.627	2.5704	14.880	8.4
29	1.0008	.....	14.178				68	2.5710	.....	62.901			
30	1.2029	.....	18.716				69	2.5816	.....	62.920			
31	1.2023	.....	18.811				70	2.7118	.....	68.636			
32	1.2036	.....	18.818	1.2026	4.4506	+1.3	71	2.7142	.....	68.423			
33	1.2018	.....	18.793				72	2.7126	.....	68.338	2.7090	16.217	9.2
34	1.2024	.....	18.770				73*	2.6876	.....	68.962			
35	1.4009	.....	23.678				74	2.6976	.....	68.344			
36	1.3997	.....	23.780										
37	1.4035	.....	23.843	1.4007	5.6376	+2.1							
38	1.3973	.....	23.774										
39	1.4023	.....	23.879										

*Omit in average.

TABLE 24.—Schoder and Turner, Sharp-Crested Weirs, Series P.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet. per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	1.0045	.....	14.213				7	2.0442	.....	42.847			
2	1.0095	.....	14.389	1.0075	3.3976	+0.9	8	2.0404	.....	42.625	2.0413	10.1301	4.3
3	1.0085	.....	14.413				9	2.0392	.....	42.775			
4	1.0086	.....	14.617				10	2.0422	.....	44.588			
5	1.0075	.....	14.567	1.0102	3.4732	2.7	11	2.0465	.....	45.034	2.0416	10.6524	9.7
6	1.0144	.....	14.788				12	2.0362	.....	45.237			

TABLE 25.—Dawson, (Nearly) Sharp-Crested Weir, Series I.

No. of run.	Observed head, in feet, by float-gauge.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head, in feet, by float-gauge.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.096	0.4472	0.096	0.1060	7.1	21	1.3955	23.79			
2	0.187	1.185				22	1.397	23.80	1.397	5.642	2.6
3	0.185	1.194	0.186	0.2819	5.5	23	1.398	23.83			
4	0.267	2.006				24	1.613	29.87			
5	0.267	2.027	0.267	0.4778	4.0	25	1.612	29.88	1.612	7.078	3.9
6	0.443	4.249	0.443	1.007	2.5	26	1.811	35.31			
7	0.459	4.477	0.459	1.061	2.4	27	1.806	35.29			
8	0.542	5.752	0.542	1.363	2.6	28	1.818	35.78	1.813	8.416	3.5
9	0 559	6.035	0.559	1.430	2.7	29	1.817	35.69			
10	0.789	10.05				30	2.021	42.27			
11	0.789	10.04	0.789	2.380	2.0	31	2.023	42.30	2.023	10.021	4.6
12	0.789	10.04				32	2.025	42.28			
13	0.910	12.39				33	2.398	54.51			
14	0.910	12.42	0.910	2.940	1.7	34	2.398	56.15			
15	1.000	14.31				35	2.403	54.93	2.394	12.979	5.2
16	1.001	14.40	1.000	3.403	2.2	36	2.376	53.50			
17	1.000	14.37				37	2.453	58.03	2.453	13.751	7.5
18	1.203	19.06				38	2.543	62.00			
19	1.202	18.86	1.202	4.498	2.5	39	2.548	59.10	2.546	14.349	6.1
20	1.2005	19.02									

TABLE 26.—Dawson, Sharp-Crested Weir, Series II.

No. of run.	Observed head, by float-gauge, in feet.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head, by float-gauge, in feet.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.111	0.5348				27	1.212	18.829			
2	0.111	0.5366	0.1110	0.1271	3.3	28	1.213	18.799	1.2123	4.4600	0.3
3	0.111	0.5375				29	1.212	18.834			
4	0.177	1.0472				30	1.372	22.748			
5	0.177	1.0500	0.1770	0.2488	0.3	31	1.373	22.880	1.3720	5.3934	0.8
6	0.177	1.0530				32	1.371	22.651			
7	0.248	1.7330				33	1.565	27.924			
8	0.248	1.7383	0.2480	0.4116	+0.1	34	1.568	27.919	1.5690	6.6275	1.3
9	0.248	1.7395				35	1.574	28.061			
10	0.317	2.5062				36	1.760	33.418			
11	0.317	2.5110	0.3170	0.5925	−0.3	37	1.761	33.558	1.7597	7.9370	2.1
12	0.317	2.5143				38	1.758	33.506			
13	0.425	3.8783				39	1.915	38.614			
14	0.425	3.8563	0.4250	0.9163	−0.7	40	1.917	38.702	1.9187	9.1600	3.5
15	0.425	3.8661				41	1.924	38.648			
16	0.546	5.6860				42	2.155	46.233			
17	0.546	5.6737	0.5460	1.3460	+0.2	43	2.151	46.101	2.1537	10.936	3.9
18	0.546	5.6503				44	2.155	46.119			
19	0.546	5.6908				45	2.352	53.015			
20	0.926	12.461				46	2.349	53.407			
21	0.928	12.538	0.9288	2.9645	−0.5	47	2.345	52.736	2.3472	12.578	5.0
22	0.927	12.382				48	2.343	53.158			
23	0.934	12.660				49	2.645	64.928			
24	1.050	15.125				50	2.652	64.983			
25	1.057	15.177	1.0520	3.5870	−0.2	51	2.655	65.465	2.6496	15.402	7.2
26	1.049	15.108				52	2.647	64.799			
						53	2.649	64.809			

TABLE 27.—DAWSON, SHARP-CRESTED WEIR (AFTER SIX WEEKS' SERVICE), SERIES III.

No. of run.	Observed head by float-gauge, in feet.	Measured discharge, in second-feet, per foot of weir.	Mean of observed heads, in feet.	Mean discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head by float-gauge, in feet.	Measured discharge, in second-feet per foot of weir.	Mean of observed heads, in feet.	Mean discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.115	0.1360				18	1.079	3.754			
2	0.115	0.1348	0.115	0.1355	+4.3	19	1.079	3.757	1.079	3.755	0.6
3	0.115	0.1358				20	1.079	3.754			
4	0.254	0.4306				21	1.368	5.39			
5	0.254	0.4310	0.254	0.4310	1.1	22	1.366	5.405	1.3655	5.395	1.5
6	0.254	0.4314				23	1.363	5.385			
7	0.330	0.638				24	1.365	5.40			
8	0.330	0.636	0.330	0.6360	0.7	25	1.680	7.425			
9	0.330	0.634				26	1.678	7.430			
10	0.437	0.960				27	1.683	7.420	1.6802	7.424	2.3
11	0.437	0.967	0.437	0.9660	0.3	28	1.680	7.420			
12	0.437	0.971				29	2.082	10.26			
13	0.524	1.281				30	2.086	10.26			
14	0.524	1.281	0.524	1.2810	1.5	31	2.087	10.44	2.0836	10.340	3.2
15	0.978	3.225				32	2.082	10.37			
16	0.978	3.223	0.978	3.2250	0.1	33	2.081	10.37			
17	0.978	3.227									

TABLE 28.—MARTIN, SHARP-CRESTED WEIRS.

No. of run.	Observed head, mean of point and plumb-bob gauges, in feet.	Measured discharge, in second-feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' Formula.	No. of run.	Observed head, mean of point and plumb bob gauges, in feet.	Measured discharge, in second-feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0 095	0.2050	0.1013	3.9	16	0.693	4.130	2.042	6.2
2	0.151	0.4167	0.2060	5.3	17	0.717	4.357	2.154	6.5
3	0.204	0.6352	0.3140	2.3	18	0.741	4.560	2.254	6.0
4	0.204	0.6302	0.3115	1.5	19	0.812	5.312	2.626	7.7
5	0.239	0.8109	0.4008	3.0	20	0.851	5.711	2.823	7.9
6	0.306	1.172	0.5793	2.7	21	0.861	5.777	2.856	7.3
7	0.339	1.363	0.6738	2.4	22	0.874	5.973	2.952	8.5
8	0.400	1.753	0.8665	2.8	23	0.910	6.341	3.134	8.4
9	0.459	2.177	1.076	3.9	24	0.910	6.349	3.138	8.5
10	0.491	2.406	1.189	3.7	25	0.914	6.424	3.176	9.1
11	0.560	2.940	1.453	4.1	26	0.915	6.431	3.179	9.0
12	0.571	3.059	1.512	5.2	27	0.960	6.988	3.454	10.2
13	0.595	3.205	1.584	3.6	28	1.029	7.676	3.795	9.1
14	0.631	3.572	1.766	5.7	29	1.052	8.000	3.954	10.0
15	0.640	3.667	1.813	6.3	30	1.064	8.032	3.970	8.6

TABLE 29.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES A.
(Runs, 1 to 42, 43 to 56, 57 to 106, 107 to 109; Date, March 21, March 26, April 2, and May 7, 1914; Water Temperature, in Degrees Fahrenheit, 33, 34, 38, 62.)

No. of run.	Observed Head, in Feet.		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	0.050	0.051	0.0858	0.0505	0.0423	11.8	54	.....	.....	1.9356	0.430	0.9554	1.7
2	0.050	0.051	0.0861	0.0505	0.0425	12.1	55	.....	.....	1.9313	0.430	0.9533	1.5
3	.....	.....	0.1057	0.060	0.0522	6.6	56	.....	.....	1.9324	0.430	0.9538	1.6
4	.....	.....	0.1055	0.060	0.0521	6.5	57	0.440	0.441	1.9988	0.4405	0.9866	1.3
5	0.070	0.071	0.1404	0.0705	0.0693	11.4	58	.....	.....	2.0663	0.450	1.0199	1.5
6	0.070	0.071	0.1379	0.0705	0.0681	9.2	59	.....	.....	2.0782	0.450	1.0258	2.1
7	0.080	0.081	0.1668	0.0805	0.0823	8.4	60	.....	.....	2.2154	0.470	1.0935	1.8
8	0.080	0.081	0.1660	0.0805	0.0819	7.9	61	.....	.....	2.2150	0.470	1.0933	1.8
9	0.091	0.093	0.2045	0.0920	0.1009	8.7	62	.....	.....	2.3239	0.486	1.1470	1.8
10	0.092	0.094	0.2088	0.0930	0.1006	6.3	63	.....	.....	2.3679	0.490	1.1687	2.3
11	.....	.....	0.2194	0.0980	0.1083	6.0	64	.....	.....	2.3647	0.490	1.1672	2.2
12	.....	.....	0.2194	0.0980	0.1083	6.0	65	.....	.....	2.5070	0.510	1.2374	2.1
13	0.111	0.113	0.2706	0.1120	0.1336	6.9	66	.....	.....	2.5145	0.510	1.2411	2.3
14	0.111	0.113	0.2694	0.1120	0.1330	6.5	67	.....	.....	2.8088	0.550	1.3864	2.1
15	.....	.....	0.2901	0.1190	0.1432	4.7	68	.....	.....	2.7985	0.549	1.3813	2.1
16	0.130	0.131	0.3303	0.1305	0.1630	3.9	69	0.580	0.582	3.0414	0.581	1.5012	2.0
17	0.130	0.131	0.3302	0.1305	0.1630	3.9	70	.....	.....	3.1806	0.598	1.5699	2.0
18	.....	.....	0.3554	0.1370	0.1754	3.9	71	.....	.....	3.1893	0.599	1.5742	2.0
19	0.150	0.152	0.4141	0.1510	0.2044	4.5	72	0.649	0.647	3.6040	0.648	1.7789	2.4
20	0.150	0.152	0.4148	0.1510	0.2047	4.7	73	.....	.....	3.6576	0.654	1.8053	2.4
21	.....	.....	0.4609	0.1640	0.2275	2.6	74	0.703	0.702	4.0571	0.7025	2.0025	2.1
22	0.170	0.171	0.4893	0.1705	0.2415	3.1	75	0.699	0.703	4.0424	0.701	1.9952	2.1
23	0.170	0.171	0.4880	0.1705	0.2409	2.6	76	0.730	0.735	4.3147	0.7325	2.1296	2.0
24	0.190	0.191	0.5760	0.1905	0.2843	2.6	77	.....	.....	4.4801	0.751	2.2113	2.1
25	0.190	0.191	0.5771	0.1905	0.2848	2.9	78	0.750	0.749	4.4577	0.7495	2.2002	1.8
26	.....	.....	0.6130	0.1990	0.3026	2.4	79	.....	.....	4.7146	0.775	2.3270	2.4
27	0.210	0.211	0.6738	0.2105	0.3326	3.4	80	0.798	0.801	4.9048	0.7995	2.4209	1.8
28	0.210	0.211	0.6685	0.2105	0.3300	2.6	81	0.798	0.799	4.9109	0.7985	2.4239	2.1
29	0.230	0.231	0.7643	0.2305	0.3772	2.4	82	0.810	0.814	5.0427	0.812	2.4890	2.1
30	0.230	0.231	0.7598	0.2305	0.3750	1.8	83	0.850	0.851	5.3925	0.8505	2.6616	2.0
31	.....	.....	0.8003	0.2380	0.3950	2.3	84	0.849	0.848	5.3990	0.8485	2.6648	2.3
32	0.250	0.251	0.8618	0.2505	0.4254	2.1	85	0.886	0.891	5.7756	0.8885	2.8507	2.3
33	0.250	0.251	0.8648	0.2505	0.4268	2.4	86	0.904	0.907	5.9325	0.9055	2.9282	2.1
34	0.270	0.271	0.9702	0.2705	0.4789	2.4	87	0.904	0.907	5.9406	0.9055	2.9322	2.1
35	0.271	0.272	0.9732	0.2715	0.4804	2.3	88	0.947	0.950	6.3944	0.9485	3.1561	2.6
36	.....	.....	1.0115	0.279	0.4993	2.0	89	0.950	0.951	6.4160	0.9505	3.1668	2.4
37	.....	.....	1.0736	0.290	0.5299	1.8	90	.....	.....	6.4236	0.950	3.1706	2.9
38	.....	.....	1.0689	0.290	0.5276	1.5	91	0.999	1.003	6.9901	1.001	3.4502	3.4
39	.....	.....	1.1841	0.310	0.5844	1.8	92	0.999	1.001	6.9604	1.000	3.4355	3.2
40	.....	.....	1.1830	0.310	0.5839	1.6	93	1.054	1.059	7.5326	1.0565	3.7179	2.9
41	.....	.....	1.2411	0.320	0.6126	1.6	94	1.104	1.106	8.0572	1.105	3.9769	2.9
42	.....	.....	1.2985	0.330	0.6409	1.5	95	1.099	1.105	8.0569	1.102	3.9767	3.2
43	.....	.....	1.2978	0.330	0.6406	1.5	96	1.134	1.138	8.4023	1.136	4.1472	2.9
44	.....	.....	1.4232	0.350	0.7025	1.8	97	1.196	1.202	9.2059	1.199	4.5438	3.9
45	.....	.....	1.4256	0.350	0.7036	2.0	98	1.200	1.198	9.1638	1.199	4.5231	3.4
46	.....	.....	1.4692	0.358	0.7252	1.6	99	1.249	1.257	9.8722	1.253	4.8727	4.4
47	.....	.....	1.5410	0.370	0.7606	1.5	100	1.257	1.261	9.8656	1.259	4.8695	3.4
48	.....	.....	1.5379	0.370	0.7591	1.3	101	1.303	1.300	10.4234	1.3015	5.1448	4.0
49	.....	.....	1.6707	0.390	0.8246	1.7	102	1.300	1.302	10.3757	1.301	5.1212	3.7
50	.....	.....	1.6702	0.390	0.8244	1.7	103	1.301	1.307	10.5625	1.304	5.2134	5.0
51	0.400	0.401	1.7335	0.4005	0.8556	1.3	104	1.334	1.330	11.0884	1.332	5.4730	7.1
52	.....	.....	1.8013	0.410	0.8891	1.6	105	1.334	1.339	11.1675	1.3365	5.5121	7.3
53	.....	.....	1.8090	0.410	0.8929	2.1	106	1.338	1.344	11.1373	1.341	5.4971	6.3

TABLE 30.—Meyer and See, Sharp-Crested Weirs, Set I, Series B.

(Date, May 11-12, 1914; Temperature of Water, 56 to 57° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge, by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0717	0.043	0.0355	19.5	20	0.393	0.392	1.6860	0.3925	0.8342	1.9
2	.....	.....	0.1182	0.059	0.0585	22.6	21	.....	.....	1.8861	0.422	0.9333	2.2
3	.....	.....	0.1832	0.084	0.0906	11.7	22	.....	.....	2.1433	0.459	1.0605	2.4
4	.....	.....	0.2370	0.102	0.1173	8.1	23	.....	.....	2.4676	0.503	1.2210	2.8
5	0.119	0.118	0.2911	0.1185	0.1440	6.0	24	.....	.....	2.8236	0.547	1.3971	3.7
6	.....	.....	0.3711	0.140	0.1836	5.3	25	.....	.....	3.2446	0.599	1.6054	4.0
7	.....	.....	0.3856	0.143	0.1908	5.9	26	.....	.....	3.6550	0.649	1.8085	3.9
8	.....	.....	0.4443	0.159	0.2198	4.1	27	.....	.....	3.8397	0.673	1.8999	3.3
9	.....	.....	0.5320	0.179	0.2632	4.4	28	.....	.....	4.0897	0.703	2.0236	3.1
10	.....	.....	0.5844	0.199	0.2892	— 2.2	29	.....	.....	4.4859	0.747	2.2196	3.2
11	.....	.....	0.7142	0.219	0.3534	3.5	30	.....	.....	4.9850	0.804	2.4666	2.7
12	.....	.....	0.8050	0.239	0.3983	2.4	31	.....	.....	5.4476	0.849	2.6955	3.5
13	.....	.....	0.9268	0.262	0.4586	2.7	32	.....	.....	5.9107	0.897	2.9246	3.4
14	.....	.....	1.0119	0.278	0.5007	2.6	33	.....	.....	6.5428	0.957	3.2374	3.8
15	.....	.....	1.1268	0.299	0.5573	2.4	34	.....	.....	7.0000	1.001	3.4636	3.9
16	.....	.....	1.2474	0.320	0.6172	2.4	35	.....	.....	8.0897	1.101	4.0028	4.0
17	.....	.....	1.3737	0.341	0.6797	2.5	36	.....	.....	9.2257	1.197	4.5649	4.7
18	.....	.....	1.4682	0.359	0.7265	1.4	37	.....	.....	9.9301	1.257	4.9135	4.7
19	.....	.....	1.6013	0.379	0.7923	2.0	38	.....	.....	11.0740	1.325	5.4795	7.9

TABLE 31.—Meyer and See, Sharp-Crested Weirs, Set I, Series C.

(Date, May 14, 1914; Temperature of Water, 50° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0992	0.053	0.0491	20.9	20	.....	.....	1.8756	0.421	0.9281	2.0
2	.....	.....	0.1053	0.058	0.0521	12.0	21	.....	.....	2.1085	0.451	1.0433	3.4
3	.....	.....	0.1507	0.071	0.0746	18.4	22	.....	.....	2.4642	0.499	1.2193	3.9
4	.....	.....	0.2320	0.094	0.1148	19.6	23	.....	.....	2.8178	0.546	1·3943	3.8
5	.....	.....	0.2618	0.110.	0.1295	6.6	24	.....	.....	3.2401	0.599	1.6032	3.8
6	.....	.....	0.3351	0.131	0.1658	5.0	25	.....	.....	3.2922	0.608	1.6290	3.2
7	.....	.....	0.3476	0.134	0.1720	5.3	26	.....	.....	3.6761	0.650	1.8190	4.2
8	.....	.....	0.4165	0.152	0.2061	4.5	27	.....	.....	4.1030	0.701	2.0302	3.9
9	.....	.....	0.4847	0.169	0.2398	3.6	28	.....	.....	4.5781	0.754	2.2653	3.9
10	.....	.....	0.5842	0.191	0.2891	4.0	29	.....	.....	5.0265	0.802	2.4871	4.0
11	.....	.....	0.6862	0.214	0.3395	3.0	30	.....	.....	5.5027	0.853	2.7228	3.8
12	.....	.....	0.7722	0.232	0.3821	2.7	31	.....	.....	6.0119	0.904	2.9747	3.9
13	.....	.....	0.8703	0.251	0.4306	2.8	32	.....	.....	6.5260	0.953	3.2291	4.2
14	0.270	0.269	0.9642	0.2695	0.4791	2.8	33	.....	.....	7.0652	1.003	3.4959	4.5
15	.....	.....	1.0825	0.291	0.5356	2.5	34	.....	.....	8.2791	1.114	4.0965	4.6
16	.....	.....	1.2526	0.321	0.6198	2.3	35	.....	.....	9.2991	1.198	4.6012	5.4
17	.....	.....	1.4363	0.352	0.7107	2.2	36	.....	.....	10.0257	1.260	4.9608	5.3
18	.....	.....	1.5539	0.371	0.7689	2.2	37	.....	.....	11.0673	1.328	5.4762	7.5
19	.....	.....	1.6317	0.384	0.8074	1.9							

TABLE 32.—Meyer and See, Sharp-Crested Weirs, Set I, Series D.
(Date, May 16, 1914; Temperature of Water, 52 to 53° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0809	0.048	0.0400	14.3	20	.....	.....	1.7887	0.410	0.8855	1.3
2	0.069	0.070	0.1412	0.0695	0.0699	14.6	21	.....	.....	2.0650	0.447	1.0223	2.7
3	.....	.....	0.1535	0.077	0.0760	6.7	22	.....	.....	2.4167	0.497	1.1964	2.5
4	.....	.....	0.2520	0.101	0.1248	16.8	23	.....	.....	2.4884	0.505	1.2319	3.1
5	.....	.....	0.3338	0.122	0.1652	16.4	24	.....	.....	2.8182	0.548	1.3951	3.3
6	.....	.....	0.3532	0.137	0.1749	3.6	25	.....	.....	3.2962	0.606	1.6318	3.9
7	.....	.....	0.4072	0.139	0.2016	16.8	26	.....	.....	3.6944	0.654	1.8289	3.8
8	.....	.....	0.4729	0.167	0.2341	3.0	27	.....	.....	4.1567	0.709	2.0578	3.5
9	.....	.....	0.5441	0.184	0.2694	2.5	28	.....	.....	4.6179	0.762	2.2861	3.2
10	.....	.....	0.6397	0.205	0.3167	2.5	29	.....	.....	5.1550	0.818	2.5520	3.6
11	.....	.....	0.7045	0.219	0.3488	2.2	30	.....	.....	5.6563	0.869	2.8002	3.8
12	.....	.....	0.7985	0.239	0.3953	1.6	31	0.918	0.920	6.1700	0.919	3.0545	4.1
13	.....	.....	0.9088	0.260	0.4499	1.9	32	.....	.....	6.8356	0.973	3.3840	5.9
14	.....	.....	1.0307	0.283	0.5102	1.8	33	.....	.....	7.5190	1.044	3.7223	4.8
15	.....	.....	1.0272	0.283	0.5085	1.4	34	.....	.....	8.2847	1.113	4.1013	4.9
16	.....	.....	1.1214	0.300	0.5551	1.4	35	1.217	1.219	9.5103	1.218	4.7081	5.2
17	.....	.....	1.2539	0.323	0.6207	1.5	36	1.300	1.293	10.5500	1.2965	5.2228	6.2
18	.....	.....	1.2459	0.325	0.6168	0.0	37	1.320	1.323	11.0585	1.3215	5.4745	8.2
19	.....	.....	1.5057	0.365	0.7454	1.5							

TABLE 33.—Meyer and See, Sharp-Crested Weirs, Set I, Series E.
(Date, May 16, 1914; Temperature of Water, 53 to 56° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0852	0.049	0.0422	16.9	20	.....	.....	1.7429	0.400	0.8624	2.4
2	.....	.....	0.1328	0.067	0.0657	18.7	21	.....	.....	1.9370	0.428	0.9584	2.8
3	.....	.....	0.1445	0.074	0.0715	6.7	22	.....	.....	1.9801	0.433	0.9798	3.3
4	.....	.....	0.2198	0.092	0.1088	17.1	23	.....	.....	2.4345	0.497	1.2046	3.2
5	.....	.....	0.2925	0.112	0.1447	16.0	24	.....	.....	2.8590	0.550	1.4145	4.1
6	.....	.....	0.3787	0.133	0.1874	16.0	25	.....	.....	3.3086	0.606	1.6371	4.2
7	.....	.....	0.4555	0.163	0.2254	2.9	26	.....	.....	3.6904	0.652	1.8260	4.2
8	.....	.....	0.5038	0.175	0.2493	2.3	27	.....	.....	4.1277	0.702	2.0424	4.3
9	.....	.....	0.5659	0.189	0.2800	2.3	28	.....	.....	4.5539	0.750	2.2533	4.2
10	.....	.....	0.6379	0.205	0.3156	2.1	29	.....	.....	4.9403	0.796	2.4445	4.5
11	.....	.....	0.7337	0.225	0.3630	2.1	30	.....	.....	5.0732	0.803	2.5102	4.8
12	.....	.....	0.7722	0.233	0.3821	2.0	31	.....	.....	5.5743	0.852	2.7582	5.3
13	.....	.....	0.8445	0.248	0.4179	1.6	32	.....	.....	6.1420	0.905	3.0391	6.0
14	.....	.....	0.9563	0.269	0.4732	1.9	33	.....	.....	6.6989	0.957	3.3146	6.3
15	.....	.....	1.0555	0.287	0.5223	2.0	34	.....	.....	7.1750	1.000	3.5502	6.6
16	.....	.....	1.1599	0.305	0.5739	2.3	35	1.101	1.103	8.2950	1.102	4.1044	6.5
17	.....	.....	1.3011	0.330	0.6438	2.0	36	1.201	1.203	9.5693	1.202	4.7349	7.9
18	.....	.....	1.4221	0.350	0.7037	2.1	37	.....	.....	10.4253	1.263	5.1585	9.1
19	.....	.....	1.5710	0.375	0.7773	1.6	38	.....	.....	11.1783	1.305	5.5311	11.4

TABLE 34.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES F.
(Date, May 19, 1914; Temperature of Water, 58 to 59° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0685	0.039	0.0338	32.0	20	.....	.....	1.4917	0.358	0.7370	3.3
2	.....	.....	0.0735	0.042	0.0363	26.4	21	.....	.....	1.7787	0.400	0.8788	4.3
3	.....	.....	0.1134	0.058	0.0560	20.4	22	.....	.....	2.0862	0.443	1.0367	5.0
4	.....	.....	0.1839	0.081	0.0909	18.4	23	.....	.....	2.5365	0.504	1.2532	5.2
5	.....	.....	0.2486	0.100	0.1228	16 6	24	.....	.....	2.9884	0.559	1.4765	6.1
6	.....	.....	0.3355	0.122	0.1658	16.9	25	.....	.....	3.1026	0.573	1·5329	6.1
7	.....	.....	0.3338	0.131	0.1649	4.4	26	.....	.....	3.3245	0.597	1.6425	6.9
8	.....	.....	0.4026	0.138	0.1989	16.5	27	.....	.....	3.7884	0.647	1.8717	8.0
9	.....	.....	0.4802	0.167	0.2373	4.4	28	.....	.....	4.2083	0.699	2.0792	6.8
10	.....	.....	0.5301	0.179	0.2619	3.8	29	.....	.....	4.6783	0·747	2.3089	7.4
11	.....	.....	0.6118	0.198	0.3023	3.0	30	.....	.....	5.1835	0.802	2.5610	7.1
12	.....	.....	0.7260	0.222	0.3587	3.0	31	.....	.....	5.6770	0.848	2.8048	7.9
13	.....	.....	0.8142	0.239	0.4023	3.4	32	.....	.....	6.3048	0.904	3.1150	8.8
14	.....	.....	0.9228	0.260	0.4559	3.3	33	.....	.....	6.7761	0.949	3.3479	8.7
15	.....	.....	1.0446	0.282	0.5161	3.5	34	.....	.....	7.3858	1.001	3.6491	9.4
16	.....	.	1.0877	0.290	0.5374	3.3	35	.....	.....	7.8850	1.045	3.8957	9.5
17	.....	..	1.1398	0.298	0.5631	3.9	36	.....	.....	8.4902	1.095	4.1948	9.9
18	.....	...	1.2575	0.318	0.6213	4.0	37	.....	.....	9.4581	1.171	4.6730	10.7
19	.....	.....	1.3778	0.338	0.6807	4.0	38	.....	.....	11.2151	1.286	5.5411	14.1

TABLE 35.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES G.
(Date, May 19, 1914; Temperature of Water, 59 to 62° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0842	0.050	0.0416	11.8	20	.....	.....	1.7562	0.396	0.8681	4.6
2	.....	.....	0.1408	0.070	0.0696	12.8	21	.....	.....	2.1561	0.448	1.0658	6.7
3	.....	.....	0.1695	0.083	0.0838	5.3	22	.....	.....	2.5174	0.495	1.2441	7.3
4	.....	.....	0.2078	0.095	0.1027	5.3	23	.....	.....	2.9800	0.552	1.4731	7.9
5	.....	.....	0.2732	0.115	0.1350	3.9	24	.....	.....	3.4317	0.602	1.6963	9.1
6	.....	.....	0.3281	0.130	0.1622	3 9	25	.....	.....	3.8702	0.651	1.9131	9.4
7	.....	.....	0.4022	0.150	0.1988	2.7	26	.....	.....	4.3106	0.697	2.1308	10.0
8	.....	.....	0.4844	0.170	0.2394	2.6	27	.....	.....	4.4302	0.709	2.1899	10.1
9	.....	.....	0.5684	0.190	0.2810	1.9	28	.....	.....	4.8624	0.755	2.4036	10.0
10	.....	.....	0.6711	0.211	0.3317	2.8	29	.....	.....	5.2641	0.795	2.6021	10.2
11	.....	.....	0.7650	0.230	0.3782	3.0	30	.....	.....	5.8713	0.850	2.9023	11.2
12	.....	.....	0.8748	0.252	0.4324	2.6	31	.....	.....	6.6473	0.919	3.2859	12.0
13	.....	.....	0.9776	0.270	0.4832	3.4	32	.....	.....	6.9851	0.946	3.4528	12.7
14	.....	.....	1.0954	0.291	0.5415	3.6	33	.....	.....	7.6575	1.004	3.7852	13.0
15	.....	.....	1.2124	0.311	0.5993	3.8	34	.....	.....	8.4390	1.065	4.1715	14.0
16	.....	.....	1.3214	0.328	0.6532	4.4	35	.....	.....	8.8501	1.097	4.3747	14.3
17	.....	.....	1.4387	0.349	0.7112	3.6	36	.....	.....	10.2516	1.200	5.0675	15.8
18	.....	.....	1.5781	0.368	0.7801	4.9	37	.....	.....	11.1942	1.249	5.5335	19.0
19	.....	.....	1.6524	0.382	0.8168	3.9							

TABLE 36.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES B.
(Date, April 17, 1914; Temperature of Water, 40 to 47° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0886	0.052	0.0437	10.3	19	0.405	0.407	1.9950	0.406	0.9847	14.2
2	0.057	0.059	0.1007	0.058	0.0497	6.6	20	0.446	0.449	2.3505	0.4475	1.1602	16.2
3	.....	.....	0.1398	0.0725	0.0690	6.3	21	0.495	0.498	2.7947	0.4965	1.3794	18.5
4	0.077	0.079	0.1527	0.078	0.0754	4.0	22	0.514	0.517	2.9751	0.5155	1.4684	19.1
5	0.091	0.092	0.1979	0.0915	0.0977	6.0	23	0.550	0.554	3.3173	0.552	1.6374	19.8
6	0.110	0.111	0.2633	0.1105	0.1300	6.1	24	0.599	0.603	3.7984	0.601	1.8748	20.9
7	0.120	0.122	0.2967	0.121	0.1464	4.4	25	0.647	0.651	4.3441	0.649	2.1442	23.2
8	.....	.....	0.3491	0.134	0.1723	5.3	26	0.703	0.709	4.9827	0.706	2.4594	24.4
9	.....	.....	0.4242	0.152	0.2094	6.0	27	0.739	0.744	5.4403	0.7415	2.6852	26.4
10	.....	.....	0.4204	0.152	0.2075	5.2	28	0.779	0.784	5.8853	0.7815	2.9049	26.4
11	.....	.....	0.4980	0.170	0.2458	5.2	29	0.823	0.829	6.5500	0.826	3.2329	29.4
12	0.199	0.200	0.6346	0.1995	0.3132	5.5	30	0.843	0.849	6.7581	0.846	3.3357	28.8
13	0.243	0.244	0.8732	0.2435	0.4310	5.7	31	0.892	0.898	7.3247	0.895	3.6153	28.1
14	0.243	0.244	0.8699	0.2435	0.4294	5.3	32	0.997	1.003	8.8876	1.000	4.3867	31.8
15	0.246	0.248	0.8915	0.247	0.4400	7.7	33	1.060	1.067	9.9294	1.0635	4.9010	32.6
16	0.290	0.291	1.1493	0.2905	0.5673	8.9	34	1.092	1.099	10.4648	1.0955	5.1652	35.2
17	0.350	0.352	1.5490	0.351	0.7646	10.5	35	1.119	1.124	10.9522	1.1215	5.4058	36.8
18	0.398	0.401	1.9199	0.3995	0.9476	12.7	36	1.122	1.125	10.8983	1.1235	5.3792	35.7

TABLE 37.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES C.
(Date, April 18, 1914; Temperature of Water, 48 to 52° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0726	0.044	0.0358	15.9	20	.....	.....	2.0215	0.422	0.9978	9.5
2	.....	.....	0.1208	0.065	0.0596	7.9	21	.....	.....	2.4400	0.474	1.2043	11.0
3	.....	.....	0.1388	0.068	0.0685	15.9	22	0.505	0.506	2.6944	0.5055	1.3299	11.1
4	.....	.....	0.1738	0.084	0.0858	5.8	23	0.545	0.547	3.0764	0.546	1.5184	12.9
5	.....	.....	0.2348	0.108	0.1159	5.3	24	0.590	0.592	3.4850	0.591	1.7201	13.7
6	.....	.....	0.3014	0.123	0.1488	8.4	25	0.652	0.654	4.1167	0.653	2.0319	15.6
7	.....	.....	0.3772	0.142	0.1862	4.4	26	0.699	0.701	4.6117	0.700	2.2762	16.7
8	0.157	0.158	0.4405	0.1575	0.2174	4.4	27	0.738	0.739	5.0139	0.7385	2.4748	16.9
9	0.179	0.181	0.5360	0.180	0.2646	4.0	28	0.750	0.753	5.1776	0.7515	2.5556	17.7
10	.....	.....	0.5422	0.181	0.2676	4.4	29	0.805	0.808	5.8032	0.8065	2.8643	18.8
11	.....	.....	0.6445	0.203	0.3181	4.5	30	0.851	0.853	6.3553	0.852	3.1368	19.8
12	0.218	0.219	0.7196	0.2185	0.3552	4.5	31	0.902	0.905	7.0000	0.9035	3.4551	20.7
13	0.239	0.240	0.8299	0.2395	0.4096	5.0	32	0.930	0.933	7.3333	0.9315	3.6196	20.9
14	.....	.....	0.9548	0.262	0.4713	5.5	33	0.949	0.952	7.6059	0.9505	3.7541	21.5
15	.....	.....	1.0623	0.280	0.5243	6.3	34	1.002	1.004	8.2954	1.003	4.0944	22.4
16	.....	.....	1.2072	0.305	0.5958	6.3	35	1.003	1.006	8.3237	1.0045	4.1084	22.5
17	0.344	0.346	1.4583	0.345	0.7198	6.6	36	1.096	1.098	9.6167	1.097	4.7466	24.0
18	0.377	0.378	1.6890	0.3775	0.8337	8.1	37	1.151	1.153	10.4424	1.152	5.1542	25.2
19	0.420	0.422	2.0200	0.421	0.9970	9.7	38	1.175	1.179	10.8777	1.177	5.3690	26.3

TABLE 38.—Meyer and See, Sharp-Crested Weirs, Set II, Series D.

(Date, April 25, 1914; Temperature of Water, 48° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0904	0.052	0.0446	12.7	20	.....	.....	1.7512	0.393	0.8644	5.5
2	.....	.....	0.1098	0.061	0.0542	7.9	21	0.416	0.417	1.9050	0.4165	0.9403	5.0
3	.....	.....	0.1432	0.073	0.0707	7.9	22	.....	.....	1.9379	0.421	0.9565	5.3
4	.....	.....	0.1768	0.085	0.0873	5.5	23	.....	.....	2.1867	0.454	1.0794	6.1
5	.....	.....	0.1982	0.091	0.0978	7.1	24	.....	.....	2.5574	0.500	1.2623	7.1
6	.....	.....	0.2729	0.114	0.1347	5.0	25	.....	.....	2.9734	0.550	1.4676	8.1
7	.....	.....	0.2982	0.121	0.1472	4.8	26	.....	.....	3.8446	0.651	1.8976	8.5
8	.....	.....	0.3448	0.134	0.1702	4.0	27	.....	.....	4.2895	0.698	2.1172	8.9
9	.....	.....	0.4387	0.158	0.2165	3.4	28	.....	.....	4.7066	0.740	2.3228	9.5
10	.....	.....	0.5191	0.177	0.2562	3.2	29	.....	.....	5.0020	0.768	2.4689	10.1
11	.....	.....	0.5757	0.190	0.2842	3.1	30	.....	.....	5.3074	0.797	2.6196	10.5
12	.....	.....	0.6782	0.212	0.3347	3.1	31	0.855	0.857	5.9427	0.856	2.9332	11.3
13	.....	.....	0.7723	0.231	0.3812	3.1	32	.....	.....	6.4963	0.905	3.2064	11.9
14	0.235	0.236	0.7992	0.2355	0.3945	3.7	33	.....	.....	7.0613	0.952	3.4853	12.7
15	.....	.....	0.8799	0.251	0.4343	3.7	34	0.996	0.997	7.5994	0.9965	3.7509	13.4
16	.....	.....	0.9927	0.272	0.4900	3.7	35	.....	.....	7.6829	1.003	3.7921	13.5
17	.....	.....	1.1862	0.305	0.5855	4.4	36	.....	.....	8.8510	1.098	4.3687	14.2
18	0.321	0.322	1.2812	0.3215	0.6324	4.2	37	1.223	1.226	10.6090	1.2245	5.2364	16.1
19	.....	.....	1.4324	0.345	0.7070	4.8	38	.....	.....	11.1553	1.259	5.5060	17.1

TABLE 39.—Meyer and See, Sharp-Crested Weirs, Set II, Series E.

(Date, May 2, 1914; Temperature of Water, 49 to 53° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0784	0.047	0.0387	13.5	20	.....	.....	2.0629	0.441	1.0182	4.5
2	.....	.....	0.1418	0.071	0.0700	11.3	21	.....	.....	2.1689	0.456	1.0705	4.5
3	.....	.....	0.1999	0.090	0.0987	9.7	22	.....	.....	2.5191	0.500	1.2434	5.5
4	.....	.....	0.2280	0.100	0.1125	6.9	23	.....	.....	2.9249	0.553	1.4437	5.5
5	.....	.....	0.2748	0.113	0.1356	7.3	24	.....	.....	3.2914	0.598	1.6246	5.5
6	.....	.....	0.3442	0.132	0.1699	6.3	25	.....	.....	3.7508	0.650	1.8513	6.1
7	.....	.....	0.4053	0.148	0.2000	5.5	26	0.671	0.673	3.9536	0.672	1.9514	6.3
8	.....	.....	0.4835	0.168	0.2386	4.0	27	.....	.....	4.2379	0.704	2.0917	6.3
9	.....	.....	0.5441	0.182	0.2686	3.9	28	.....	.....	4.7782	0.760	2.3584	6.9
10	.....	.....	0.5830	0.190	0.2878	4.4	29	.....	.....	5.1688	0.800	2.5512	7.1
11	.....	.....	0.6827	0.212	0.3370	3.7	30	0.850	0.851	5.6920	0.8505	2.8095	7.4
12	.....	.....	0.7670	0.230	0.3786	3.1	31	.....	.....	6.3306	0.908	3.1247	8.4
13	0.253	0.252	0.8894	0.2525	0.4390	3.9	32	.....	.....	6.7783	0.950	3.3456	8.5
14	.....	.....	0.9868	0.271	0.4871	3.9	33	1.000	1.003	7.3738	1.0015	3.6396	9.0
15	.....	.....	1.1593	0.301	0.5722	4.0	34	1.030	1.032	7.7005	1.031	3.8008	9.0
16	.....	.....	1.3469	0.333	0.6648	3.7	35	1.106	1.107	8.6109	1.1065	4.2502	9.7
17	.....	.....	1.5088	0.360	0.7447	3.6	36	1.227	1.230	10.1767	1.2285	5.0230	10.8
18	.....	.....	1.7883	0.403	0.8827	3.7	37	1.290	1.294	11.1877	1.292	5.5220	12.9
19	.....	.....	1.7950	0.403	0.8860	4.0							

TABLE 40.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES F.
(Date, May 5, 1914; Temperature of Water, 54 to 58° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0716	0.046	0.0353	6.9	20	0.452	0.451	2.0938	0.4515	1.0335	2.1
2	.....	.....	0.0904	0.055	0.0446	3.4	21	0.464	0.463	2.1827	0.4635	1.0773	2.8
3	.....	.....	0.1265	0.068	0.0624	5.7	22	.....	.....	2.5050	0.506	1.2364	3.2
4	.....	.....	0.2172	0.098	0.1072	4.8	23	.....	.....	2.9005	0.557	1.4316	3.4
5	.....	.....	0.2910	0.120	0.1436	3.7	24	0.602	0.600	3.2512	0.601	1.6047	3.4
6	.....	.....	0.3595	0.141	0.1774	0.7	25	.....	.....	3.6394	0.647	1.7963	3.6
7	0.146	0.145	0.3815	0.1455	0.1883	1.6	26	0.703	0.701	4.1079	0.702	2.0276	3.4
8	.....	.....	0.4517	0.162	0.2230	2.6	27	0.706	0.705	4.1402	0.7055	2.0435	3.6
9	.....	.....	0.5226	0.180	0.2579	1.5	28	.....	.....	4.5158	0.747	2.2289	3.4
10	.....	.....	0.6159	0.201	0.3040	1.5	29	0.804	0.802	5.0433	0.803	2.4893	3.9
11	.....	.....	0.7114	0.220	0.3511	2.3	30	0.856	0.854	5.5648	0.855	2.7467	4.4
12	.....	.....	0.7998	0.239	0.3948	1.5	31	.....	.....	6.0316	0.900	2.9771	4.7
13	.....	.....	0.9275	0.263	0.4578	1.8	32	0.949	0.947	6.5477	0.948	3.2318	5.2
14	.....	.....	1.0268	0.282	0.5068	1.6	33	1.004	1.002	7.1537	1.003	3.5309	5.5
15	0.296	0.294	1.0972	0.295	0.5416	1.5	34	1.023	1.021	7.3729	1.022	3.6391	5.8
16	0.301	0.300	1.1308	0.3005	0.5581	1.6	35	1.099	1.097	8.2591	1.098	4.0766	6.5
17	.....	.....	1.3521	0.339	0.6674	1.6	36	1.200	1.198	9.4953	1.199	4.6867	7.1
18	.....	.....	1.5954	0.378	0.7875	1.8	37	1.316	1.314	11.2197	1.315	5.5379	10.3
19	.....	.....	1.8650	0.420	0.9205	1.5							

TABLE 41.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES G.
(Date, May 7, 1914; Temperature of Water, 60 to 62° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0897	0.052	0.0443	11.8	20	.....	.....	1.8700	0.419	0.9230	2.1
2	.....	.....	0.1099	0.056	0.0542	22.4	21	.....	.....	2.1531	0.457	1.0627	3.1
3	.....	.....	0.1781	0.078	0.0879	21.2	22	0.500	0.498	2.4868	0.499	1.2274	4.7
4	.....	.....	0.1770	0.083	0.0874	9.7	23	.....	.....	2.6096	0.517	1.2880	4.0
5	.....	.....	0.2638	0.110	0.1302	7.3	24	.....	.....	2.8533	0.550	1.4083	3.7
6	.....	.....	0.3263	0.128	0.1611	5.5	25	.....	.....	3.2910	0.604	1.6244	3.9
7	.....	.....	0.3900	0.145	0.1925	4.7	26	.....	.....	3.6728	0.652	1.8128	3.4
8	.....	.....	0.4700	0.164	0.2320	4.8	27	.....	.....	4.1142	0.699	2.0307	4.4
9	.....	.....	0.4862	0.168	0.2400	4.7	28	.....	.....	4.6040	0.756	2.2724	3.9
10	0.181	0.18	0.5394	0.1805	0.2662	4.0	29	.....	.....	5.0631	0.803	2.4990	4.4
11	.....	.....	0.6283	0.200	0.3101	4.0	30	.....	.....	5.0982	0.807	2.5164	4.2
12	.....	.....	0.7188	0.220	0.3548	3.2	31	.....	.....	5.5274	0.851	2.7282	4.4
13	.....	.....	0.8234	0.241	0.4064	3.1	32	0.899	0.898	6.0215	0.8985	2.9721	4.8
14	.....	.....	0.9290	0.262	0.4585	2.4	33	.....	.....	6.5688	0.949	3.2422	5.3
15	.....	.....	1.0493	0.284	0.5179	2.8	34	.....	.....	7.1583	1.002	3.5332	5.8
16	.....	.....	1.1411	0.301	0.5632	2.6	35	.....	.....	8.2768	1.099	4.0853	6.6
17	.....	.....	1.2465	9.318	0.6153	2.9	36	.....	.....	9.4508	1.196	4.6647	7.1
18	.....	.....	1.3548	0.337	0.6687	2.8	37	.....	.....	9.6834	1.213	4.7795	7.4
19	.....	.....	1.6260	0.381	0.8026	2.4	38	.....	.....	11.0886	1.306	5.4731	10.1

TABLE 42.—Jones, Series A, Sharp Crest.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0198	0.0206	0.02013	0.0202	0.009934	4.34	22	0.6471	0.6478	3.5401	0.6474	1.7542	1.10
2	0.0303	0.0303	0.04102	0.0303	0.02033	15.82	23	0.7450	0.7458	4.3930	0.7458	2.1768	1.57
3	0.0400	0.0408	0.05943	0.0404	0.02945	8.90	24	0.8513	0.8528	5.3719	0.8520	2.6620	1.65
4	0.0490	0.0483	0.07717	0.0487	0.03824	6.87	25	0.9463	0.9468	6.3219	0.9465	3.1327	2.17
5	0.0595	0.0598	0.10240	0.0596	0.05074	4.73	26	1.0527	1.0565	7.4879	1.0527	3.7105	3.17
6	0.0690	0.0698	0.12772	0.0694	0.06329	3.96	27	1.1510	1.1518	8.6220	1.1514	4.2725	3.86
7	0.0800	0.0803	0.15964	0.0801	0.07911	4.80	28	1.1502	1.1518	8.6031	1.1510	4.2632	3.68
8	0.0895	0.0895	0.19098	0.0895	0.09464	6.15	29	1.3600	1.3588	11.260	1.3594	5.5798	5.72
9	0.1005	0.1003	0.22838	0.1004	0.11317	6.90	30	1.3583	1.3603	11.217	1.3583	5.5585	5.44
10	0.1185	0.1183	0.2960	0.1184	0.14668	8.11	31	0.0196	0.0196	0.01732	0.0196	0.008583	−6.07
11	0.1395	0.1388	0.3530	0.1392	0.1749	1.10	32	0.0405	0.0400	0.05718	0.0403	0.02833	5.17
12	0.1590	0.1583	0.4345	0.1587	0.2153	2.28	33	0.0808	0.0808	0.1609	0.0808	0.07973	4.26
13	0.1785	0.1783	0.5162	0.1784	0.2558	1.95	34	0.1008	0.1003	0.2258	0.1006	0.1119	5.27
14	0.1995	0.1995	0.6090	0.1995	0.3018	1.72	35	0.1193	0.1188	0.2968	0.1191	0.1471	7.45
15	0.2495	0.2498	0.8497	0.2496	0.4211	1.42	36	0.1313	0.1315	0.3302	0.1314	0.1636	3.16
16	0.2800	0.2803	1.0090	0.2801	0.5000	1.27	37	0.0308	0.0313	0.04043	0.0310	0.02003	10.24
17	0.3513	0.3513	1.4131	0.3513	0.7002	0·98	38	0.0413	0.0418	0.06085	0.0415	0.03015	7.11
18	0.3998	0.3993	1.7096	0.3996	0.8472	0.71	39	0.0528	0.0533	0.08502	0.0530	0.04213	3.69
19	0.4483	0.4483	2.0313	0.4483	1.0066	0.71	40	0.0613	0.0620	0.10643	0.0616	0.05274	3.59
20	0.4983	0.4983	2.3944	0.4983	1.1865	1.29	41	0.1388	0.1388	0.3564	0.1388	0.1766	2.43
21	0.5498	0.5503	2.7684	0.5500	1.3718	1.00	42	0.1793	0.1793	0.5195	0.1793	0.2574	1.82

TABLE 43.—Jones, Series B, Up-Stream Edge Rubbed Once Over with Fine Emery Paper Under Pressure of About 5 Pounds.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0198	0.0204	0.01998	0.0201	0.009901	4.33	17	0.3475	0.3481	1.3969	0.3478	0.6922	1.34
2	0.0288	0.0294	0.03890	0.0291	0.019276	16.60	18	0.3975	0.3986	1.7006	0.3980	0.8427	0.79
3	0.0388	0.0394	0.05642	0.0391	0.02796	8.58	19	0.4488	0.4497	2.0403	0.4492	1.0110	0.85
4	0.0488	0.0494	0.07750	0.0491	0.03840	5.99	20	0.4990	0.5039	2.4013	0.5015	1.1899	0.62
5	0.0588	0.0589	0.10018	0.0586	0.04964	5.08	21	0.5996	0.5996	3.1602	0.5996	1.5660	1.29
6	0.0683	0.0689	0.12647	0.0686	0.06267	4.75	22	0.6982	0.6990	3.9803	0.6986	1.9724	1.44
7	0.0788	0.0794	0.1578	0.0791	0.07820	5.55	23	0.7979	.....	4.8653	0.7979	2 4109	1.58
8	0.0908	0.0914	0.1962	0.0911	0.09722	6.18	24	0.9000	0.9044	5.8873	0.9022	2.9174	2.24
9	0.0983	0.0989	0.2209	0.0986	0.1095	6.21	25	0.9978	0.9979	6.8849	0.9978	3.4117	2.80
10	0.1183	0.1189	0.2875	0.1186	0.1425	4.78	26	1.1054	.....	8.1130	1.1054	4.0203	3.86
11	0.1398	0.1404	0.3637	0.1401	0.1802	3.21	27	1.1069	.....	8.1048	1.1069	4.0161	3.56
12	0.1588	0.1594	0.4390	0.1591	0.2175	2.93	28	1.1926	.....	9.1394	1.1926	4.5290	4.43
13	0.1805	0.1816	0.5296	0.1810	0.2624	2.34	29	1.1914	.....	9.0947	1.1914	4.5068	4.08
14	0.2005	0.2019	0.6175	0.2012	0.3060	1.83	30	1.3559	1.3579	11.2252	1.3569	5.5625	5.69
15	0.2478	0.2489	0.8468	0.2483	0.4196	1.85	31	1.3568	1.3579	11.2489	1.3573	5.5743	5.86
16	0.2988	0.2994	1.1154	0.2991	0.5527	1.47							

TABLE 44.—Jones, Series C, Crest Rounded 0.0065 Inch.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0196	0.0193	0.01847	0.0195	0.009153	0.94	17	0.3481	0.3478	1.3966	0.3480	0.6921	1.24
2	0.0306	0.0302	0.04107	0.0304	0.020352	15.31	18	0.4037	0.4033	1.7430	0.4035	0.8637	1.20
3	0.0401	0.0400	0.05833	0.0401	0.028903	8.13	19	0.4535	0.4525	2.0734	0.4530	1.0274	1.19
4	0.0491	0.0493	0.07783	0.0492	0.038568	6.13	20	0.4981	0.4975	2.3953	0.4978	1.1870	1.49
5	0.0606	0.0598	0.10357	0.0602	0.051324	4.34	21	0.6011	.....	3.1814	0.6011	1.5765	1.58
6	0.0706	0.0701	0.13068	0.0704	0.064758	4.12	22	0.6435	.....	3.9381	0.6435	1.9515	.....
7	0.0801	0.0798	0.15978	0.0800	0.079168	5.07	23	0.7990	0.7984	4.8940	0.7987	2.4252	2.06
8	0.0896	0.0893	0.18972	0.0895	0.094014	5.44	24	0.8992	0.8990	5.8615	0.8991	2.9046	2.31
9	0.1016	0.1018	0.23338	0.1017	0.1156	7.02	25	0.9969	0.9981	6.8750	0.9975	3.4068	2.69
10	0.1211	0.1213	0.3075	0.1212	0.1524	8.47	26	0.9966	0.9980	6.8860	0.9973	3.4123	2.89
11	0.1388	0.1383	0.3603	0.1386	0.1785	3.90	27	1.0988	1.0998	7.9891	1.0993	3.9590	3.15
12	0.1596	0.1595	0.4407	0.1596	0.2184	2.87	28	1.0982	.....	8.0249	1.0982	3.9767	3.77
13	0.1811	0.1813	0.5313	0.1812	0.2633	2.53	29	1.1941	1.1933	9.1801	1.1937	4.5490	4.74
14	0.2018	0.2021	0.6217	0.2019	0.3081	2.02	30	1.1941	.....	9.1361	1.1941	4.5273	4.27
15	0.2525	0.2531	0.8668	0.2528	0.4295	1.47	31	1.3572	1.3594	11.2161	1.3583	5.5580	5.43
16	0.3006	0.3008	1.1246	0.3007	0.5573	1.49	32	1.3578	1.3574	11.2048	1.3576	5.5524	5.42

TABLE 45.—Jones, Series D, Crest Rounded 0.0075 Inch.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0405	0.0407	0.06024	0.0406	0.02985	9.73	19	0.1605	0.1597	0.4450	0.1601	0.2205	3.37
2	0.0580	0.0582	0.10067	0.0581	0.04989	6.99	20	0.1790	0.1792	0.5250	0.1791	0.2601	3.05
3	0.0792	0.0795	0.16115	0.07935	0.07986	7.31	21	0.2002	0.2002	0.6163	0.2002	0.3054	2.88
4	0.1004	0.1008	0.2312	0.1006	0.1146	7.80	22	0.2510	0.2507	0.8600	0.2508	0.4261	1.89
5	0.1395	0.1395	0.3648	0.1395	0.1808	4.21	23	0.3000	0.2997	1.1207	0.2998	0.5553	1.59
6	0.1790	0.1784	0.5235	0.1787	0.2594	3.10	24	0.3505	0.3497	1.4067	0.3501	0.6971	1.06
7	0.2507	0.2512	0.8623	0.2509	0.4273	2.11	25	0.4020	0.4017	1.7316	0.4018	0.8581	1.17
8	0.0185	0.0182	0.01618	0.0184	0.008018	.....	26	0.4528	.....	2.0669	0.4528	1.0242	0.95
9	0.0301	0.0297	0.04052	0.0299	0.02008	16.62	27	0.4990	.....	2.4010	0.4990	1.1898	1.36
10	0.0405	0.0402	0.05973	0.0404	0.02960	9.45	28	0.6021	.....	3.1838	0.6021	1.5777	1.40
11	0.0490	0.0487	0.07793	0.0488	0.03859	7.19	29	0.6973	.....	3.9774	0.6973	1.9710	1.65
12	0.0610	0.0607	0.10608	0.0608	0.05257	5.31	30	0.7984	.....	4.8737	0.7984	2.4151	1.66
13	0.0705	0.0702	0.13200	0.0704	0.06541	5.16	31	0.9016	.....	5.8917	0.9016	2.9196	2.42
14	0.0792	0.0787	0.15917	0.0790	0.07887	6.67	32	0.9987	.....	6.9020	0.9987	3.4202	2.91
15	0.0893	0.0887	0.1920	0.0890	0.09514	7.60	33	1.1009	.....	8.0530	1.1009	3.9906	3.75
16	0.1013	0.1007	0.2330	0.1010	0.1155	8.05	34	1.1984	.....	9.1962	1.1984	4.5571	4.31
17	0.1180	0.1177	0.2870	0.1178	0.1422	5.56	35	1.3556	.....	11.2195	1.3556	5.5597	5.77
18	0.1412	0.1405	0.3707	0.1408	0.1837	4.43	36	1.3557	.....	11.2253	1.3557	5.5626	5.88

TABLE 46.—JONES, SERIES E, CREST ROUNDED 0.0120 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0209	0.0204	0.02131	0.0206	0.01056	7.26	16	0.3009	0.3004	1.1292	0.3006	0.5596	1.93
2	0.0289	0.0287	0.03893	0.0288	0.01929	18.55	17	0.3544	0.3534	1.4341	0.3539	0.7106	1.36
3	0.0389	0.0389	0.05767	0.0389	0.02858	11.85	18	0.4022	0.4017	1.7361	0.4020	0.8703	1.36
4	0.0524	0.0524	0.08578	0.0524	0.04251	6.43	19	0.4523	0.4519	2.0699	0.4521	1.0257	1.33
5	0.0604	0.0609	0.10660	0.0606	0.05283	6.34	20	0.4993	0.4997	2.3382	0.4995	1.1587	....
6	0.0694	0.0689	0.13092	0.0692	0.06488	7.02	21	0.6015	.....	3.1784	0.6015	1.5750	1.38
7	0.0799	0.0799	0.1624	0.0799	0.08048	7.00	22	0.6981	.....	3.9947	0.6981	1.9795	1.91
8	0.0909	0.0909	0.1986	0.0909	0.09841	7.80	23	0.8003	.....	4.9110	0.8003	2.4336	2.08
9	0.1036	0.1033	0.2414	0.1034	0.1196	8.03	24	0.8979	.....	5.8668	0.8979	2.9072	2.61
10	0.1176	0.1177	0.2860	0.1176	0.1417	5.51	25	1.0010	.....	6.9394	1.0010	3.4375	3.08
11	0.1406	0.1399	0.3680	0.1402	0.1824	4.35	26	1.0985	.....	8.0405	1.0985	3.9844	3.92
12	0.1624	0.1617	0.4530	0.1620	0.2245	3.41	27	1.0992	.....	8.0411	1.0992	3.9847	3.84
13	0.1812	0.1809	0.5333	0.1810	0.2643	3.08	28	1.3542	.....	11.2088	1.3542	5.5544	5.84
14	0.2016	0.2009	0.6220	0.2012	0.3082	2.56	29	1.3545	.....	11.1943	1.3545	5.5472	5.68
15	0.2514	0.2506	0.8640	0.2512	0.4281	2.12							

TABLE 47.—JONES, SERIES F, CREST ROUNDED 0.0165 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0208	0.0212	0.02090	0.0210	0.01035	2.18	16	0.3048	0.3042	1.1488	0.3045	0.5693	1.73
2	0.0291	0.0297	0.03935	0.0294	0.01950	16.15	17	0.3463	0.3472	1.3966	0.3468	0.6921	1.77
3	0.0418	0.0424	0.06453	0.0421	0.03198	11.20	18	0.4048	0.4044	1.7562	0.4046	0.8703	1.57
4	0.0523	0.0527	0.08713	0.0525	0.04318	7.79	19	0.4589	.....	2.1264	0.4589	1.0537	1.79
5	0.0608	0.0614	0.1094	0.0611	0.05421	7.79	20	0.4972	.....	2.4025	0.4972	1.1905	1.97
6	0.0710	0.0712	0.1366	0.0711	0.06769	7.22	21	0.6014	.....	3.1858	0.6014	1.5787	1.64
7	0.0813	0.0812	0.1681	0.0812	0.08330	8.13	22	0.7044	.....	4.0474	0.7044	2.0056	1.88
8	0.0948	0.0947	0.2131	0.0948	0.10560	8.64	23	0.7992	.....	4.9024	0.7992	2.4293	2.11
9	0.1050	0.1047	0.2694	0.1048	0.1335	....	24	0.8993	.....	5.8608	0.8993	2.9043	2.27
10	0.1278	0.1282	0.3257	0.1280	0.1614	5.83	25	0.9974	.....	6.8977	0.9982	3.4181	3.08
11	0.1408	0.1414	0.3740	0.1411	0.1853	4.98	26	0.9978	.....	6.8941	0.9978	3.4163	2.93
12	0.1600	0.1607	0.4487	0.1604	0.2223	3.93	27	1.0976	.....	8.0201	1.0976	3.9743	3.79
13	0.1816	0.1817	0.5383	0.1816	0.2667	3.49	28	1.3556	.....	11.1800	1.3556	5.5401	5.59
14	0.2018	0.2022	0.6283	0.2020	0.3113	2.98	29	1.3534	.....	11.2079	1.3534	5.5510	5.94
15	0.2518	0.2524	0.8703	0.2521	0.4313	2.30							

TABLE 48.—Jones, Series G, Crest Rounded 0.0415 ($\frac{1}{24}$) Inch.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0204	0.0205	0.02252	0.0204	0.01116	15.00	16	0.3032	0.3023	1.1544	0.3028	0.5720	3.08
2	0.0309	0.0310	0.04582	0.0310	0.02270	24.90	17	0.3459	.....	1.4073	0.3459	0.6974	2.95
3	0.0406	0.0407	0.06480	0.0406	0.03211	17.80	18	0.4088	0.4079	1.7984	0.4084	0.8912	2.54
4	0.0509	0.0510	0.08783	0.0510	0.04352	13.45	19	0.4548	0.4539	2.1063	0.4544	1.0438	2.33
5	0.0589	0.0585	0.1075	0.0587	0.05327	12.45	20	0.4944	.....	2.4009	0.4944	1.1887	2.69
6	0.0672	0.0675	0.1316	0.0674	0.06521	11.90	21	0.6023	.....	3.2163	0.6023	1.5938	2.39
7	0.0790	0.0790	0.1665	0.0790	0.08251	11.58	22	0.7036	.....	4.0651	0.7036	2.0144	2.50
8	0.0916	0.0917	0.2116	0.0916	0.10485	13.55	23	0.7955	.....	4.9044	0.7955	2.4303	2.86
9	0.1029	0.1030	0.2470	0.1030	0.1224	11.15	24	0.9001	.....	5.8921	0.9001	2.9198	2.68
10	0.1229	0.1225	0.3130	0.1227	0.1551	8.38	25	1.0020	.....	6.9651	1.0020	3.4515	3.34
11	0.1434	0.1430	0.3903	0.1432	0.1934	7.16	26	1.0025	.....	6.9747	1.0025	3.4562	3.40
12	0.1619	0.1615	0.4653	0.1617	0.2306	6.52	27	1.0957	.....	8.0207	1.1055	3.9746	4.07
13	0.1801	0.1792	0.5400	0.1796	0.2676	5.60	28	1.3507	.....	11.1517	1.3507	5.5261	5.70
14	0.1994	0.1993	0.6273	0.1994	0.3108	4.83	29	1.3531	.....	11.2170	1.3531	5.5585	6.05
15	0.2511	0.2513	0.8800	0.2512	0.4361	4.03							

TABLE 49.—Jones, Series H, Sharp Crest.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge of Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0172	0.0178	0.01502	0.0175	0.007447	.....	20	0.5021	.....	2.4126	0.5021	1.1961	0.95
2	0.0287	0.0293	0.03681	0.0290	0.01825	11.00	21	0.6014	.....	3.1713	0.6014	1.5722	1.23
3	0.0376	0.0383	0.05314	0.03808	0.02635	6.81	22	0.6011	.....	3.1657	0.6011	1.5695	1.13
4	0.0492	0.0498	0.07737	0.0495	0.03836	4.61	23	0.6995	.....	3.9824	0.6995	1.9752	1.38
5	0.0582	0.0588	0.09833	0.0585	0.04875	3.45	24	0.7000	.....	3.9825	0.7000	1.9757	1.30
6	0.0682	0.0688	0.1243	0.0685	0.06163	3.23	25	0.8007	.....	4.8867	0.8007	2.4227	1.55
7	0.0772	0.0783	0.1515	0.0778	0.07511	3.95	26	0.8014	.....	4.8951	0.8014	2.4269	1.58
8	0.0872	0.0883	0.1833	0.0878	0.09088	4.91	27	0.9020	.....	5.8583	0.9020	2.9045	1.82
9	0.1007	0.1013	0.2274	0.1010	0.1127	5.42	28	0.9001	.....	5.8510	0.9001	2.9008	2.01
10	0.1192	0.1198	0.2975	0.1195	0.1475	7.20	29	1.0007	.....	6.8843	1.0007	3.4131	2.39
11	0.1384	0.1391	0.3588	0.1388	0.1779	3.31	30	0.9998	.....	6.8802	0.9998	3.4111	2.46
12	0.1592	0.1598	0.4398	0.1595	0.2180	2.78	31	1.1103	.....	8.0091	1.1103	3.9709	2.19
13	0.1787	0.1793	0.5213	0.1790	0.2584	2.46	32	1.1111	.....	8.0260	1.1111	3.9792	2.02
14	0.1982	0.1988	0.6050	0.1985	0.2999	1.83	33	1.3552	.....	11.1315	1.3552	5.5188	5.05
15	0.2497	0.2503	0.8513	0.2500	0.4221	1.42	34	1.3567	.....	11.1287	1.3567	5.5174	4.86
16	0.3002	0.3008	1.1180	0.3005	0.5543	1.04	35	1.1123	.....	8.1513	1.1123	4.0413	3.45
17	0.3477	0.3478	1.3880	0.3477	0.6882	0.81	36	1.1120	.....	8.1646	1.1120	4.0479	3.66
18	0.4072	0.4063	1.7326	0.4068	0.8689	0.57	37	1.3564	.....	11.1772	1.3564	5.5419	5.36
19	0.4568	.....	2.0883	0.4568	1.0353	0.70	38	1.3567	.....	11.2098	1.3567	5.5576	5.62

TABLE 50.—Weber, Series II, Roughness, 96 Grooves per Inch for 4 Inches Below Crest Level.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.02122	0.0212	0.01057	2.87	20	2.3940	0.4989	1.1910	1.50
2	0.04078	0.0316	0.02032	8.62	21	3.1607	0.6006	1.5725	1.45
3	0.05703	0.0402	0.02842	5.88	22	4.0206	0.7033	2.0003	1.85
4	0.07453	0.0485	0.03714	4.42	23	4.8491	0.7952	2.4101	2.07
5	0.1023	0.0604	0.05097	5.24	24	5.7576	0.9029	2.8616	.....
6	0.1278	0.0702	0.06368	2.81	25	6.9682	1.0059	3.4633	3.09
7	0.1543	0.0788	0.07688	4.36	26	6.9623	1.0052	3.4569	3.01
8	0.1820	0.0877	0.09068	4.86	27	8.0369	1.1011	3.9905	3.71
9	0.2215	0.0997	0.1104	5.34	28	8.0279	1.0997	3.9860	3.80
10	0.2970	0.1224	0.1480	3.79	29	11.1538	1.3547	5.5381	5.48
11	0.3723	0.1430	0.1855	3.00	30	11.2054	1.3547	5.5637	5.96
12	0.4397	0.1605	0.2189	2.24	31	1.4309	0.3545	7.1242	1.37
13	0.5357	0.1835	0.2667	1.91	32	2.0564	0.4519	1.0231	1.10
14	0.6093	0.2000	0.3034	1.88	33	3.1439	0.5978	1.5626	1.53
15	0.8337	0.2470	0.4151	1.54	34	5.9942	0.9121	2.9777	2.65
16	1.1016	0.2975	0.5485	1.52	35	5.9843	0.9116	2.9728	2.58
17	1.3882	0.3480	0.6912	1.11	36	11.2082	1.3569	5.5651	5.75
18	1.7599	0.4072	0.8756	1.24	37	11.1925	1.3560	5.5573	5.69
19	2.0839	0.4572	1.0367	0.70					

TABLE 51.—Weber, Series III, Roughness, 48 Grooves per Inch for 4 Inches Below Crest Level.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0234	0.0233	0.0249	0.0234	0.01243	4.28	18	0.4059	.....	1.7386	0.4059	0.8663	0.59
2	0.0301	0.0300	0.03723	0.0300	0.01859	7.46	19	0.4579	.....	2.0904	0.4579	1.0415	0.94
3	0.0389	0.0393	0.05377	0.0391	0.02684	4.24	20	0.5515	.....	2.7749	0.5515	1.3826	1.38
4	0.0484	0.0488	0.07293	0.0486	0.03641	2.04	21	0.5512	.....	2.7679	0.5512	1.3791	1.20
5	0.0584	0.0588	0.09646	0.0586	0.04816	1.95	22	0.6475	.....	3.5271	0.6475	1.7556	1.19
6	0.0689	0.0693	0.1224	0.0691	0.06111	1.025	23	0.6965	.....	3.9506	0.6965	1.9664	1.59
7	0.0782	0.0785	0.1492	0.0784	0.07449	1.92	24	0.7960	.....	4.8425	0.7960	2.4104	1.92
8	0.0889	0.0893	0.18192	0.0891	0.09081	2.54	25	0.7965	.....	4.8447	0.7965	2.4115	1.87
9	0.0979	0.0983	0.25886	0.0981	0.1293	2.64	26	0.8964	.....	5.6753	0.8964	2.8249	....
10	0.1197	0.1198	0.29130	0.1197	0.1454	5.23	27	0.8956	.....	5.7909	0.8956	2.8825	2.13
11	0.1404	0.1403	0.3580	0.1404	0.1787	2.00	28	0.9993	.....	6.8921	0.9993	3.4306	3.13
12	0.1587	0.1598	0.4357	0.1592	0.2175	2.84	29	0.9996	.....	6.8737	0.9996	3.4215	2.81
13	0.1849	0.1843	0.5381	0.1846	0.2686	1.71	30	1.0956	.....	7.9558	1.0956	3.9601	3.70
14	0.1987	0.1988	0.6000	0.1987	0.2995	1.56	31	1.0940	.....	7.9340	1.0940	3.9492	3.65
15	0.2509	0.2513	0.8503	0.2511	0.4243	1.26	32	1.3561	.....	11.1489	1.3561	5.5495	5.53
16	0.3009	0.3008	1.1144	0.3009	0.5561	1.18	33	1.3562	.....	11.1219	1.3562	5.5360	5.26
17	0.3494	.....	1.3908	0.3494	0.6940	0.89	34	1.3582	.....	11.1643	1.3582	5.5571	5.43

TABLE 52.—Weber, Series IV, Roughness, 24 Grooves per Inch for 4 Inches Below Crest Level.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.01754	0.0185	0.008759	4.52	16	1.1220	0.2997	0.5597	2.44
2	0.03557	0.0277	0.01776	15.7	17	1.4242	0.3523	0.7105	2.04
3	0.05700	0.0395	0.02846	8.88	18	1.7793	0.4082	0.8876	2.21
4	0.08087	0.0507	0.04038	6.10	19	2.0614	0.4512	1.0284	1.89
5	0.10653	0.0612	0.05320	5.22	20	2.3749	0.4950	1.1848	2.16
6	0.13420	0.0712	0.06702	5.94	21	3.1463	0.5969	1.5684	2.12
7	0.16210	0.0802	0.08095	6.57	22	4.0899	0.7093	2.0388	2.49
8	0.18904	0.0887	0.09440	7.31	23	4.8903	0.7985	2.4378	2.60
9	0.2226	0.0987	0.1116	7.65	24	5.8543	0.8991	2.9184	2.80
10	0.29167	0.1174	0.14565	8.66	25	6.9450	1.0028	3.4621	3.64
11	0.36433	0.1397	0.18175	4.59	26	8.0050	1.0945	3.9945	4.51
12	0.43367	0.1574	0.21634	3.99	27	7.9669	1.0941	3.9715	4.36
13	0.5127	0.1765	0.2558	3.60	28	11.2351	1.3565	5.6007	6.45
14	0.6083	0.1985	0.3035	3.05	29	11.2307	1.3563	5.5985	6.44
15	0.8450	0.2476	0.4215	2.71	30	11.2193	1.3548	5.5929	6.51

TABLE 53.—Weber, Series V, Roughness, 24 Grooves per Inch for 4 Inches + 15 Grooves per Inch for 8 Inches.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.01536	0.0172	0.00767	2.13	16	1.0856	0.2932	0.5423	2.59
2	0.03770	0.0294	0.01883	12.14	17	1.2583	0.3232	0.6285	2.71
3	0.05303	0.0372	0.02649	10.88	18	1.7275	0.3994	0.8628	2.65
4	0.07580	0.0484	0.03786	6.77	19	2.0184	0.4444	1.0082	2.20
5	0.09763	0.0566	0.04877	8.76	20	3.2269	0.6045	1.6118	2.98
6	0.12006	0.0654	0.05997	7.69	21	4.0857	0.7062	2.0408	3.27
7	0.14987	0.0759	0.07486	7.50	22	4.9007	0.7970	2.4479	3.27
8	0.18216	0.0864	0.09099	7.59	23	5.8314	0.8930	2.9128	3.65
9	0.22000	0.0973	0.10989	8.73	24	6.8183	0.9866	3.4057	4.37
10	0.29867	0.1182	0.1492	10.27	25	6.8001	0.9857	3.3966	4.23
11	0.37267	0.1416	0.1867	4.95	26	8.5903	1.1443	4.2968	5.27
12	0.45167	0.1616	0.2256	4.29	27	8.5807	1.1437	4.2861	5.23
13	0.52767	0.1799	0.2636	3.74	28	11.1424	1.3494	5.5656	6.62
14	0.61000	0.1986	0.3047	3.40	29	11.1985	1.3531	5.5910	6.78
15	0.83567	0.2456	0.4174	2.96					

TABLE 54.—Weber, Series VI, Roughness, 24 Grooves per Inch for 4 Inches Below Crest Level. (Duplicate of Series III.)

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.1838	0.0889	0.0918	3.96	11	2.3109	0.4913	1.1537	0.65
2	0.2804	0.1156	0.1400	6.95	12	3.0825	0.5936	1.5389	1.05
3	0.3510	0.1378	0.1752	2.88	13	3.8715	0.6876	1.9328	1.81
4	0.4427	0.1616	0.2210	2.17	14	4.7210	0.7882	2.3570	1.15
5	0.5290	0.1826	0.2641	1.61	15	5.7382	0.8923	2.8648	2.06
6	0.8293	0.2469	0.4140	1.32	16	6.8565	0.9993	3.4231	2.90
7	1.0590	0.2912	0.5287	1.05	17	6.8481	1.0014	3.4189	2.46
8	1.4266	0.3556	0.7122	0.86	18	9.0407	1.1919	4.5135	4.16
9	1.6970	0.3996	0.8472	0.71	19	11.2945	1.3622	5.6387	6.51
10	2.0125	0.4476	1.0047	0.92	20	11.2953	1.3629	5.6391	6.43

APPENDIX B

TABLE 55.—Velocities in Approach Channels (at Head Gauge Station Unless Otherwise Noted).

(Data refer to conditions approximately as for the volumetrically measured runs noted, Column (1). Head, h, determined during current meter observations. Mean velocity, v, for entire channel obtained by volumetric measurements, $\frac{Q}{A}$, corrected to agree with head, h. Other velocities by current meter. Corrected values (Columns (5), (6), and (7)), obtained from current meter vertical traverses divided by ratio, $\frac{v_m}{v}$, (see Figs. 33 to 47). Head, in feet; velocities, in feet per second.)

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel, vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level. (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Schoder and Turner, Series "A", Weir, 7.53 Feet High.													
17- 18	0.562	0.1745	0.190	0.30	0.29	0.166	39- 42	1.412	0.636	0.624	1.30	1.15	0.540
19- 23	0.760	0.269	0.272	0.56	0.50	0.245	43- 47	1.709	0.828	0.88	1.56	1.40	0.705
24- 28	0.879	0.331	0.338	0.63	0.60	0.302	48- 52	1.913	0.971	1.00	1.90	1.75	0.785
29- 33	1.000	0.402	0.429	0.75	0.70	0.364	53- 57	2.157	1.130	1.135	2.17	1.95	0.880
34- 38	1.197	0.510	0.513	1.00	0.92	0.450	59- 63	2.440	1.329	1.32	2.54	2.04	1.10
Series "B", Weir, 5.50 Feet High.													
18	0.55	0.226	0.22	0.36	0.31	0.21	45- 49	1.71	1.045	1.01	1.88	1.50	0.90
19- 23	0.76	0.347	0.35	0.61	0.54	0.32	50- 54	1.91	1.215	1.20	1.96	1.69	1.03
24- 28	0.88	0.428	0.42	0.71	0.61	0.39	55- 59	2.16	1.418	1.42	2.04	1.91	1.23
29- 33	1.00	0.518	0.54	0.80	0.73	0.48	60- 64	2.28	1.53	1.56	2.22	1.96	1.35
34- 38	1.205	0.664	0.71	1.07	0.91	0.61	65- 69	2.40	1.67	1.64	2.56	2.12	1.48
39- 44	1.416	0.826	0.89	1.27	1.12	0.75							
Series "C", Weir, 5.34 Feet High.													
76- 79	1.519	0.992	0.99	1.27	1.22	0.93	110-114	2.2626	1.63	1.48	4.13	2.89	1.05
95- 99	2.2492	1.56	1.65	2.05	1.99	1.38	115-119	2.2546	1.52	1.37	1.50	1.12	1.69
100-104	2.2296	1.885	.....	5.08*	4.71	.....	125-127	2.5350	1.875	1.95	2.88	2.75	1.47
105-109	2.2905	1.76	1.55	5.44	3.82	0.86							

*As observed; eddy at bottom.

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values. At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).			True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values. At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Series "D", Weir, 7.50 Feet High.													
31- 35	1.0018	0.396	0.449	0.78	0.67	0.395	71- 75	2.2454	1.173	1.13	1.62	1.49	1.08
36- 40	1.2008	0.512	0.508	0.90	0.76	0.472	76- 80	2.2454	1.179	1.143	1.74	1.57	1.06
41- 45	1.3981	0.635	0.634	1.00	0.85	0.596	81- 85	2.2518	1.178	1.160	1.44	1.46	1.10
46- 50	1.6029	0.769	0.764	1.26	1.04	0.710	86- 90	2.2501	1.168	1.205	1.74	1.21	1.15
51- 55	1.8052	0.889	0.885	1.59	1.30	0.789	91- 95	3.2478	1.141	1.105	0.78	0.63	1.80
56- 60	1.9995	1.020	1.05	1.84	1.76	0.898	96-100	2.2491	1.235	1.309	3.32	2.88	0.75
61- 65	2.2565	1.227	1.35	2.25	2.08	0.973	101-105	2.5100	1.433	1.58	2.60	2.36	1.125
66- 70	2.2555	1.165	1.10	1.40	1.22	1.155	106-111	2.7612	1.592	1.680	2.63	2.40	1.30
Series "L", Weir, 7.50 Feet. High.													
23- 27	0.9014	0.340	0.394	0.59	0.54	0.316	33- 37	2.2268	1.210	1.383	2.36	1.96	0.99
28- 32	1.6032	0.761	0.768	1.49	1.20	0.66							
Series "M", Weir, 7.50 Feet High.													
28- 32	0.9981	0.394	0.409	0.87	0.76	0.345	53- 57	2.0008	1.032	0.927	2.42	2.10	0.75
33- 37	1.1994	0.510	0.542	1.14	0.99	0.43	58- 62	2.2140	1.181	1.135	2.50	2.08	0.92
38- 42	1.4076	0.638	0.636	1.43	1.19	0.537	63- 67	2.4084	1.325	1.293	2.72	2.33	1.00
43- 47	1.5989	0.762	0.782	1.68	1.39	0.628	68- 72	2.5318	1.412	1.341	3.07	2.61	0.96
48- 52	1.8005	0.892	0.878	2.03	1.67	0.70							
Schoder and Turner, Series "K", Weir, 5.50 Feet High.													
25- 27	0.8024	0.380	0.337	0.46	0.48	0.368	61- 65	1.9984	1.260	1.05	2.13	1.36	1.22
28- 30	0.8986	0.444	0.443	0.56	0.57	0.425	66- 70	2.0014	1.255	1.17	1.19	0.66	1.47
31- 35	0.9995	0.514	0.514	0.69	0.65	0.49	71- 75	1.9972	1.390	1.33	4.08	3.07	0.77
36- 40	1.2054	0.661	0.614	1.01	0.92	0.60	76- 80	1.9927	1.365	1.29	3.26	2.62	0.91
41- 45	1.3976	0.811	0.762	1.29	1.08	0.74	81- 85	1.9819	1.326	1.49	2.45	1.98	1.09
46- 50	1.6009	0.976	0.912	1.58	1.24	0.89	86- 90	2.1968	1.49	1.504	2.32	2.03	1.28
51- 55	1.8135	1.147	1.128	1.63	1.44	1.04	91- 95	2.4140	1.683	1.733	2.53	2.26	1.44
56- 60	2.0111	1.314	1.30	1.98	1.72	1.17	96-100	2.6564	1.898	1.848	2.94	2.70	1.50
Series "O", Weir, 5.50 Feet High.													
25- 29	1.0022	0.517	0.508	0.78	0.72	0.48	50- 54	2.0019	1.320	1.272	2.49	1.91	1.11
30- 34	1.2017	0.662	0.604	1.20	1.03	0.58	55- 59	2.1997	1.500	1.449	2.77	2.20	1.22
35- 39	1.4000	0.816	0.793	1.56	1.23	0.71	60- 64	2.4200	1.700	1.750	2.98	2.53	1.34
40- 44	1.6034	0.980	0.938	1.87	1.44	0.85	65- 69	2.5591	1.830	1.750	3.24	2.70	1.43
45- 49	1.8106	1.151	1 080	2.27	1.71	0.92	70- 74	2.6901	1.962	2.12	3.50	2.98	1.47

TABLE 55—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Series "J", Weir, 4.0 Feet High.													
23- 25	0.8002	0.495	0.486	0.56	0.56	0.484	59- 63	1.9948	1.593	1.520	2.29	1.36	1.706
26- 28	0.9016	0.584	0.559	0.68	0.68	0·566	64- 68	2.0032	1.591	1.580	1.21	0.85	1.96
29- 33	1.0002	0.672	0.649	0.78	0.77	0.647	69- 73	1.9950	1.756	1.606	5.10	2.98	1.12
34- 38	1.2015	0.856	0.819	1.12	0.97	0.820	74- 78	2.0019	1.695	1.631	3.48	2.45	1.11
39- 43	1.3998	1.046	1.009	1.42	1.15	1.008	79- 83	2.2090	1.870	1.843	2.57	2.25	1.695
41- 48	1.6026	1.244	1.141	1.76	1.43	1.164	84- 88	2.4069	2.088	2.12	2.72	2.55	1.80
49- 53	1.7991	1.444	1.388	2.01	1.68	1.34	89- 93	2.7012	2.397	2.345	3.17	3.00	1.98
54- 58	2.0027	1.655	1.614	2.29	1.93	1.516							
Schoder and Turner, Series "I", Weir 3.0 Feet High.													
23- 25	0.8002	0.630	0.612	0.70	0.66	0.623	59- 63	1.9958	1.948	2.065	1.78	1.13	2.49
26- 28	0.9041	0.742	0.716	0.83	0.78	0.728	64- 68	2.0055	1.976	1.86	2.54	1.85	2.07
29- 33	1.0006	0.846	0.835	0.96	0.91	0.827	69- 73	2.0047	2.217	2.11	4.62	3.53	1.95
34- 38	1.2031	1.070	1.026	1.30	1.15	1.06	74- 78	2.0024	2.104	1.87	4.09	3.11	1.43
39- 43	1.3986	1.293	1.238	1.58	1.35	1.265	79- 83	2.1999	2.260	2.185	2.89	2.57	2.04
44- 48	1.6042	1.531	1.465	1.92	1.68	1.46	84- 88	2.4128	2.517	2.445	3.24	2.90	2.20
49- 53	1.7955	1.762	1.761	2.40	2.03	1.60	89- 93	2.6708	2.842	2.810	3.77	3.36	2.38
54- 58	2.0079	2.024	2.03	2.60	2.33	1.82							
Series "P", Weir 2.56 Feet High.													
1- 3	1.005	0.965	0.91	1.44	1.02	0.943	7- 9	2.025	2.240	2.24	2.66	2.33	2.16
4- 6	1.013	0.994	0.81	2.27	1.68	0.72	10-12	2.022	2.355	2.27	4.55	2.76	2.01
Series "H", Weir, 2.0 Feet High.													
22-24	0.8018	0.872	0.844	0.98	0.93	0.852	53-57	1.5982	2.015	1.889	2.45	2.08	1.966
25-27	0.9016	1.012	0.964	1.27	1.06	0.993	58-62	1.8063	2.331	2.235	2.75	2.37	2.30
28-32	1.0027	1.155	1.095	1.44	1.20	1.134	63-67	2.0100	2.637	2.585	3.19	2.75	2.52
33-37	1.0020	1.145	1.040	1.22	0.98	1.229	68-72	1.9971	2.558	2.545	2.04	1.48	3.64
38-42	1.0023	1.180	1.152	2.10	1.63	0.958	73-77	2.0034	2.967	2.475	6.60	4.00	1.92
43-47	1.2026	1.435	1.344	1.68	1.48	1.40	78-82	2.1989	2.975	3.165	3.53	3 24	2.68
48-52	1.3998	1.721	1.640	2.00	1.69	1.75	83-87	2.4810	3.363	3.63	3.74	3.67	2.98
Schoder and Turner, Series "G", Weir, 1.50 Feet High.													
23- 24	0.5002	0.601	0.611	0.71	0.63	0.598	56- 60	1.6017	2.430	2.342	2.84	2.45	2.43
25- 27	0.8024	1.083	1.096	1.19	1.11	1.087	61- 65	1.8036	2.777	2.684	3.80	2.82	2.72
28- 30	0.9084	1.257	1.211	1.42	1.25	1.26	66- 70	1.9938	3.114	3.026	4.57	3.25	3.04
31- 35	1.0025	1.418	1.376	1.61	1.43	1.414	71- 75	2.0004	3.387	3.510	4.34	4.42	2.03
36- 40	0.9986	1.492	1.346	3.11	2.30	0.955	76- 80	1.9827	3.146	3.280	0.72	1.92	4.78
41- 45	1.0018	1.415	1.240	1.64	1.07	1.650	81- 85	2.2026	3.490	3.357	4.40	3.59	3.34
46- 50	1.2072	1.760	1.705	2.02	1.77	1.755	86- 90	2.4041	3.845	3.825	4.90	4.04	3.845
51- 55	1.4065	2.097	2.027	2.43	2.14	2.05							

TABLE 55—(*Continued*).

Runs No.	Heads on weir, (*h*).	Velocities in Approach Channel.					Runs No.	Heads on weir, (*h*).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Value.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean above crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Series "F", Weir, 1.00 Feet High.													
20	0.3998	0.620	0.597	0.73	0.69	0.591	52- 56	1.3994	2.660	2.67	3.10	2.73	2.55
22- 23	0.5008	0.818	0.810	0.97	0.87	0.782	57- 61	1.6026	3.06	3.18	3.46	3.09	2.80
24- 25	0.7013	1.225	1.238	1.15	1.36	1.137	62- 66	1.8031	3.50	3.53	4.18	3.67	3.21
26- 28	0.7990	1.428	1.51	1.76	1.50	1.367	67- 71	2.0055	3.90	3.99	4.38	4.11	3.48
29- 31	0.9052	1.643	1.67	2.02	1.78	1.526	72- 76	2.0040	4.07	3.73	6.32	4.44	3.33
32- 36	1.0034	1.845	1.88	2.18	1.92	1.775	77- 81	1.9950	3.99	4.72	1.94	3.56	4.84
37- 41	1.0018	1.890	1.875	2.84	2.19	1.598	82- 86	2.2122	4.28	4.44	4.78	4.53	3.77
42- 46	1.0027	1.845	2.03	1.09	1.49	2.204	87- 91*	2.3616	4.72	4.59	5.35	5.05	3.97
47- 51	1.2042	2.255	2.25	2.64	2.35	2.156							
Schoder and Turner, Series "N" Weir, 0.75 Feet High.													
21	0.2022	0.324	0.318	0.35	0.34	0.319	51- 55	0.9888	2.218	2.265	0.91	1.64	2.90
22	0.2537	0.437	0.448	0.47	0.46	0.429	56- 60	1.2088	2.64	2.62	2.77	2.72	2.51
23	0.2988	0.537	0.533	0.58	0.57	0.524	61- 65	1.4083	3.11	3.17	3.37	3.20	2.91
24	0.3485	0.650	0.674	0.77	0.70	0.627	66- 70	1.6037	3.55	3.615	3.60	3.65	3.39
25	0.4020	0.770	0.803	0.89	0.82	0.744	71- 75	1.8187	3.99	4.21	3.51	4.04	3.88
26- 29	0.4532	0.892	0.928	1.00	0.93	0.868	76- 80	2.0016	4.43	4.675	4.27	4.55	4.12
30- 31	0.5008	0.996	0.988	1.19	1.05	0.963	81- 85	2.0092	4.63	4.93	4.53	4.77	4.36
32- 34	0.7034	1.478	1.560	1.70	1.56	1.41	86- 90	2.0168	4.36	4.35	3.81	4.22	4.71
35- 37	0.8026	1.710	1.825	2.03	1.79	1.62	91- 95	2.2045	4.86	5.05	4.49	4.96	4.55
38- 40	0.9032	1.950	2.025	2.24	2.05	1.85	(Reset)	2.1907	4.82	5.01	.....	4 89	4.63
41- 45	1.0025	2.178	2.28	2.44	2.27	2.04	96-100	2.3326	5.21	5.38	4.47	5.35	4.77
46- 50	1.0001	2.202	2.616	2.41	2.28	2.10							
Series "E", Weir, 0.50 Feet High.													
16	0.2088	0.472	0.568	0.52	0.49	0.467	33- 37	1.2056	3.345	3.182	3.97	3.55	2.88
18	0.3018	0.733	0.825	0.80	0.75	0.723	38- 42	1.4109	3.956	3.815	4.53	4.08	3.57
20	0.4020	1.020	1.090	1.31	1.13	0.932	43- 47	1.5912	4.460	4.380	5.04	4.55	4.18
22- 24	0.7990	2.158	2.02	2.68	2.36	1.860	48- 52	1.8039	5.120	5.300	5.65	5.25	4.62
25- 27	0.9000	2.482	2.365	3.14	2.63	2.05	53- 57	2.0004	5.775	6.083	6.80	5.98	4.95
28- 32	1.0028	2.777	2.684	3.41	3.02	2.30							
Myer and See, Set I, Series C, Weir, 2.068 Feet High.													
29	0.803	0.869	0.87	0.92	0.92	0.92	37	1.330	1.616	1.58	1.94	1.82	1.48
33	1.001	1.135	1.165	1.16	1.17	1.12							
Set I. Series E, Weir, 1.409 Feet High.													
30	0.818	1.160	1.137	1.04	1.16	1.16	38	1.306	2.038	2.09	1.66	2.18	1.90
34	1.004	1.480	1.52	1.36	1.52	1.46							

*Original table gives data also for verticals, 0.5 ft. from either wall.

TABLE 55—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Set I, Series G, Weir, 0.755 Feet High.													
29	0.800	1.689	1.83	1.43	1.55	1.83	37	1.248	2.759	2.90	2.60	2.68	2.89
33	1.002	2.146	2.39	1.48	1.90	2.46							
J. O. Jones, Series "A", Weir, 2.50 Feet High and 2.018 Feet Long in All Series.													
18	0.401	0.294	.....	0.30	0.294	0.293	29, 30	1.3613	1.447	.....	1.79	1.71	1.31
24	0.8612	0.808	.....	0.61	0.85	0.787							
Series "C".													
18	0.3910	0.283	.....	0.27	0.288	0.283	31, 32	1.3560	1.437	.....	1.68	1.63	1.340
24	0.8825	0.833	.....	0.85	0.828	0.839							
Series "D".													
26	0.440	0.332	.....	0.29	0.283	0·341	35, 36	1.3553	1.442	.....	1.85	1.71	1.30
31, 32	0.9419	0.906	.....	0.91	0.905	0.907							
Series "E".													
19	0.456	0.351	.....	0.28	0.289	0.363	28, 29	1.3581	1.446	.....	1.77	1.68	1.32
24, 25	0.9471	0.915	.....	0.89	0.89	0.925							
Series "F".													
19	0.462	0.360	.....	0.31	0.318	0.37	28, 29	1.3562	1.440	.....	1.74	1.67	1.32
24–26	0.9299	0.889	.....	0.86	0.87	0.895							
Series "G".													
18	0.395	0.291	.....	0.29	0.287	0.291	28, 29	1.3592	1.450	.....	1.82	1.71	1.31
24–26	0.9242	0.884	.....	0.95	0.909	0.875							
Series "H".													
21, 22	0.572	0.469	.....	0.43	0.422	0.475	37–38	1.3567	1.440	.....	1.80	1.72	1.29
27–30	0.956	0.921	.....	0.92	0.926	0.917							

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level. (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
C. M. Weber, Series II, Weir, 2.50 Feet High and 2.018 Feet Long in All Series.													
19	0.450	0.343	0.365	0.25	0.283	0.355	36, 37	1.363	1.450	1.473	1.81	1.628	1.353
34- 35	0.935	0.899	0.923	0.86	0.853	0.915							
Series III.													
17	0.349	0.243	0.264	0.25	0.250	0.242	33, 34	1.357	1.439	1.445	1.86	1.625	1.341
27- 29	0.979	0.950	0.948	0.94	0.932	0.950							
Series IV.													
19, 20	0.471	0.373	0.396	0.35	0.346	0.378	28- 30	1.356	1.450	1.430	1.83	1.635	1.350
24, 25	0.942	0.913	0.882	0.85	0.881	0.927							
Series V.													
19	0.5196	0.439	0.393	0.41	0.422	0.473	28, 29	1.352	1.448	1.470	1.72	1.604	1.365
24, 25	0.9666	0.952	0.948	1.005	0.989	0.939							
Series VI.													
10, 11	0.470	0.363	0.355	0.36	0.357	0.364	19, 20	1.362	1.459	1.456	1.60	1.574	1.396
14, 15	0.854	0.797	0.802	0.67	0.689	0.832							

APPENDIX C

Velocity of Approach

This topic has been considered by many hydraulic experimenters. Seemingly, each generation in turn has studied it anew, has placed the thought on record and forthwith has abandoned it again to the realm of speculation.

A few interesting quotations from hydraulic literature follow:

Hamilton Smith states:*

"The question to be solved is, what portion of the kinetic energy, due to the head, $\frac{v^2}{2g}$, * * * produces a useful effect at the opening of discharge" (*i. e.*, above the crest-level). * * * This subject has engaged the attention of many great minds during the past two centuries, and the discussions and controversies, if collected, would fill several good-sized volumes. If the same amount of thought and time, devoted to these almost fruitless discussions, had been employed in ascertaining what the *facts* really were, we would know a good deal more about the effect of velocity of approach than we do at present."

* * * * * * * *

"It seems reasonable to suppose that the additional force produced by the velocity of approach is chiefly due to the velocity of the section of the feeding stream nearest or in line with the section of discharge where the particles of water fairly begin to form into the escaping vein. This velocity is unquestionably greater than v (the mean velocity)."

Francis states:†

"It not infrequently happens that, in consequence of the particular form of the canal leading to the weir, or from other causes, the velocity of the water in the canal is not uniform in all parts of the section; this is a frequent cause of serious error, and is often entirely overlooked. If great irregularities exist they should be removed by causing the water to pass through one or more gratings, presenting numerous small apertures equally distributed, or otherwise, as the case may require, through which the water may pass under a small head; these gratings should be placed as far from the weir as practicable."

Francis, in deriving his theoretical expression involving velocity of approach, assumes a uniform velocity throughout the cross-section of the channel of approach. This condition is, so to speak, not a natural one, as had been recognized early in the Nineteenth Century by Lesbros, Boileau, and others.

Bazin states:‡

"* * * in varying the height of a weir, we modify not only the mean velocity, v, of approach, but also the distribution of the velocities in the water above the weir, the thickness of the falling sheet, its contraction on the sill, etc."

Bazin, however, apparently made no measurements to ascertain the extent of (or the degree of constancy in the law of) the modifications in the distribution of velocities for different heights of weir.

* "Hydraulics" (1886), pp. 68–69, 84.

† "Lowell Hydraulic Experiments" (1871), p. 135.

‡ *Annales des Ponts et Chaussées*, October, 1888; *Proceedings*, Engineers' Club of Philadelphia, January, 1890, pp. 307–308.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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BASIC INFORMATION NEEDED FOR A REGIONAL PLAN*

By Harold M. Lewis,† M. Am. Soc. C. E.

As the concentration in cities has increased and the size and complexity of the individual city have grown beyond anything that could have been foreseen a few years ago, it is only natural that the scope of planning has also increased. Starting with a concentration on the civic center, planners enlarged their activities to the comprehensive city plan; then they studied the environs and grouped together two or more closely related communities in the regional plan. The State of New York has made studies for a State-wide plan and articles have been written in regard to a National plan. The writer has been asked to describe the type of material that should be assembled as a basis for determining the elements of a regional plan.

Most of the published information on city and regional planning gives the conclusions and details after the plan has been completed, or describes methods of co-operation or publicity necessary to carry out such projects. A description of the basic data required and the form of presentation is likely to be considered a matter of routine work which is probably standardized and almost surely uninteresting. When a large regional planning project is undertaken, however, the collection of this material becomes so huge a problem that it is well worth while to have it carefully planned and organized. A paper of this kind can only attempt to point out the "high spots" of the subject, it being understood that a large amount of subsidiary information must also be gathered. It is based primarly on an anlysis of what has been accomplished toward developing a regional plan for New York and its environs. Although some of the steps described may seem to be unimportant details most of them represent the result of careful experiment and, as such, should be of some value to the professional regional planner.

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† Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

It seems desirable first to give some attention to the definition of a "regional plan". By a "region" is meant a group of separate communities which form a single metropolitan district with common interests. This, as a rule, will center about a large port or manufacturing headquarters. Obviously, the "plan" should comprise only those details which have some importance in the development or welfare of the entire region, or in the promotion of the orderly and efficient development of one of its integral parts.

Mr. Edward M. Bassett has defined the city or regional plan as "the determination of land areas stamped with definite characters by law", and points out that the community or region establishes its own plan through its legislative body or bodies. This is true of the legally adopted plan which will have positive control over future growth, but that must be preceded by planning studies which should be defined in greater detail. The late Nelson P. Lewis, M. Am. Soc. C. E., in a paper* entitled "Regional Planning" has stated that when he used this term he had in mind the following type of studies:

"Larger areas must be studied and instead of beginning with present centers and providing for expansion outward, absorbing one after another of outlying towns and villages, it is time to begin at the outer edges of a great metropolitan district, to study existing communities and strive to encourage the development of their centers, to create new social, commercial, and industrial centers, and strive to protect what is worth saving from being drawn into the vortex at the big center and absorbed bodily by it, with no line of demarcation and with no barrier to the steady outward movement of population."

The writer would suggest the following definition:

"A regional plan is one for the co-ordinated physical growth of a group of communities forming a single metropolitan district, based on the most desirable and efficient development of the whole region."

A summary of the type of data required is necessarily largely a list of headings, but is important to those who are organizing a regional plan. The following classifications are essential: (1) Physical; (2) economic; (3) social; (4) legal; and, (5) financial.

A combination of graphic, statistical, and descriptive presentation is required. The physical survey is the largest and most complicated undertaking, as it includes the preparation of those maps on which the planner must base the details of the regional plan. It might be called a survey of existing conditions, both visible and officially mapped. What is included under this heading and the form in which it should be presented will be described in considerable detail. The material required under the other four headings will only be very briefly outlined. This does not mean that they are unimportant, but they are largely supplemental. Each type of investigation can be expanded into many sub-headings, some of which may lead the investigator far afield. Each should be tested by the following questions:

1.—Should this appear in a final report?
2.—Is it essential in order to arrive at conclusions that must be incorporated in the final report?
3.—Will the expense of obtaining it be practicable and justified?

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1326.

The Regional Plan of New York and Its Environs is a pioneer project in its immensity. Started in 1921, it did not reach until 1925 a stage where it was felt that surveys and the gathering of data should be subordinated to planning studies. Much information has probably been obtained that will remain buried in the files. As there was no past experience of a comparable project to serve as a guide, this was unavoidable. Other communities can save time and expense by profiting from the experience in New York. Smaller and less complicated communities will probably not require as many types of information as have proved most useful in studying this particular problem. The Committee on a Regional Plan of New York and Its Environs was organized by the Russell Sage Foundation, which is a National organization with National interests. It was justified, therefore, in doing considerable research work along such lines as:

(*a*) Highway capacities under different conditions.
(*b*) Recreation space requirement standards.
(*c*) Length of haul of motor vehicles and transit lines.
(*d*) Standards of population and housing density.
(*e*) Variation in riding habit on transit lines.
(*f*) Model plans for a neighborhood unit.

It is hoped that some of this information may be of value to other communities. It must always be used, however, with due allowance for a difference in living conditions, habits, etc.

The main sub-headings under each of the five types of studies referred to are as follows:

Physical Survey.—

1.—Topography, boundaries, and other such basic physical features as influence the plan.
2.—Population—distribution, density, and rates of growth.
3.—Communication facilities:
  (*a*) Highways;
  (*b*) Transit—rapid transit, trolleys, and motor buses;
  (*c*) Transportation system—somewhat arbitrarily defined as including the railroad, waterway, and airway systems.
4.—Parks, parkways, and other open spaces.
5.—Sanitation—water supply, sewerage, and refuse disposal.
6.—Port and industrial development (will overlap with communication facilities).
7.—Miscellaneous, including such as:
  (*a*) Zoning (land uses established by existing zoning ordinances);
  (*b*) Land values;
  (*c*) Location of public buildings;
  (*d*) Power supply facilities.

Economic Survey.—A study of the history of the principal industries within the region should determine:

1.—Trends in location.
2.—Probable space demand in the future city.
3.—Probable future relative importance.

Social Survey.—

1.—Housing.
2.—Recreation.*

* Items 2 and 4 will necessarily overlap with the "Physical Survey."

3.—Schools.
4.—Public health and sanitation.*
5.—Correctional institutions.

Legal Survey.—

1.—Ascertainment of city planning powers of the various political units.
2.—Study of new legislation needed to carry out the regional plan.

Financial Survey.—

1.—Cost of public improvements and methods of assessment.
2.—Distribution of funds available.

The first item to be determined is the area to be studied. This may vary greatly for different regions. It should be large enough to include all those communities, the business and recreational life of which are linked with the principal business center. It should include any of those suburbs from which an appreciable number of commuters travel daily to the business center. It should extend far enough to take in all industrial plants which are directly served by, and linked with, a common port. It should also contain those larger recreational spaces which are within easy reach by rail or automobile of the city or suburban dweller. The area studied in the New York Region corresponds roughly to that within a 50-mile radius of the New York City Hall. On the north it extended approximately 60 miles, on the south, about 40 miles, and on the east all of Long Island was included, although it extends about 110 miles from the center of New York City. As a general rule, the area of a regional plan may vary between that within a 10 or 50-mile radius from the metropolitan center.

Maps of the entire region should probably be prepared on at least three different scales. These maps might be defined as follows:

1.—Regional table maps, by which is meant such maps as can readily be prepared and studied on an ordinary drafting table. On a scale of 1 in. = 2 to 3 miles.
2.—Regional wall maps. On a scale of 1 in. = 1 mile for a very large region, and perhaps on the scale of 1 in. = ½ mile for smaller regions.
3.—Detail sectional maps. On a scale of 1 in. = 2 000 ft. for large regions, or about 1 in. = 1 000 ft. for smaller ones.

Where particularly difficult problems require a local study, larger maps must be prepared of the areas concerned, on scales of 1 in. = 100 to 1 000 ft.

Each of these classes of maps should contain different types of data presented in different forms.

The so-called table maps should be merely outlines on which a single type of information should be shown. On such maps should be presented population statistics, existing railroad, rapid transit, and waterway systems, parks and parkways, and all the material necessary to study the regional aspects of all the items covered by the headings listed under "Physical Survey."

The regional wall maps will serve primarily for group discussions of the main elements of the regional plan and for exhibition purposes. They should contain the principal topographic features (in the New York District, which has elevations from sea level to more than 1 600 ft., 100-ft. contours were sufficient), the principal communication and recreation facilities, and one

* Items 2 and 4 will necessarily overlap with "Physical Survey."

main feature which should be strongly emphasized. In the New York study, these maps were about 10 ft. square, and it was found desirable to have one of them to emphasize each of the following features:

1.—Highways.
2.—Railroads, including the main rapid transit systems.
3.—Parks and parkways.
4.—Sanitation, including water-sheds used for water supply and the location of sewage disposal plants.
5.—Land uses.

An excellent method of using these maps for planning studies is to have them covered with a transparent celluloid coating after all the existing features have been added. This coating can be worked upon in water or oil colors, crayons, or a soft pencil, and proposed facilities can be tentatively added in this way. Such information can be wiped off with a wet or oily rag and revised at will without any damage to the map beneath. The coating also acts as a protecting covering.

The detail sectional maps might be called the base maps of the regional plan. They should show in considerable detail the topography, street system, streams and waterway systems, transportation systems, parks and parkways, political boundaries, pier-head and bulkhead lines, and principal existing land uses. It is upon such maps that details would be worked out after their importance was determined from study of the regional wall maps. From them final sectional maps might be reproduced at smaller scale for publication. It has been found that these maps are not very suitable for discussion at conferences, as such discussion is likely to be sidetracked on detail locations which would not be apparent on maps of smaller scale. The regional importance of any proposal cannot be readily grasped unless it is on a map all parts of which can be readily seen at one instant. The 2 000-ft. scale maps prepared of the New York Region formed a map about 18 by 20 ft. in size when assembled. Such a map is effective in an exhibition, but is too large to use as a basis for serious discussion.

It was made up of enlargements from twenty-three of the standard Government topographic sheets. They were prepared at cost for the Regional Plan Committee by the U. S. Geological Survey and corrections in the shore lines, railroads, main highways, and larger park areas were incorporated. They were printed in three colors, with a few copies without the contours, some of the latter being printed in non-photographic blue. All but the outer edges of the region were covered, and as the Government sheets, 1 mile = 1 in., were the only ones available at uniform scale for a large part of the territory outside New York City, there has been considerable local demand by official bodies and individuals for copies of the sheets. Part of their cost, therefore, has been recovered by the sale of some of the copies.

Three complete maps are being prepared on copies of this base map. Each map includes six sections made up from four (in one case, three) of the 2 000-ft. scale sheets. These maps are about 6 by 7½ ft., which is about as large as can be readily used. Joining these maps with additional sections for title and legend, respectively, the 18 by 20-ft. rectangular map already referred to, was obtained.

The first of these maps showed existing land uses classified as follows:

Industrial areas.
Principal business areas (in communities of more than 25 000 population).
Close residence development.
Parks and parkways of more than 25 acres and forest and water supply reservations.
Golf and country clubs.
Cemeteries of more than 25 acres.
Large public institutional properties.
Aviation fields.

} Shown by same color with different symbols superimposed.

The classification of the first three groups was largely a matter of judgment and was intended to show the type of use which predominated in any particular area. Aeroplane maps and oblique views were found to be very useful in the preparation of this map, and it was constantly revised as more complete information was obtained. It was necessary, therefore, to use colors that could readily be changed, and flat tints, obtained by rubbing in powdered school chalk with a piece of chamois, were found to be very satisfactory. They can be readily removed with an ordinary soft eraser.

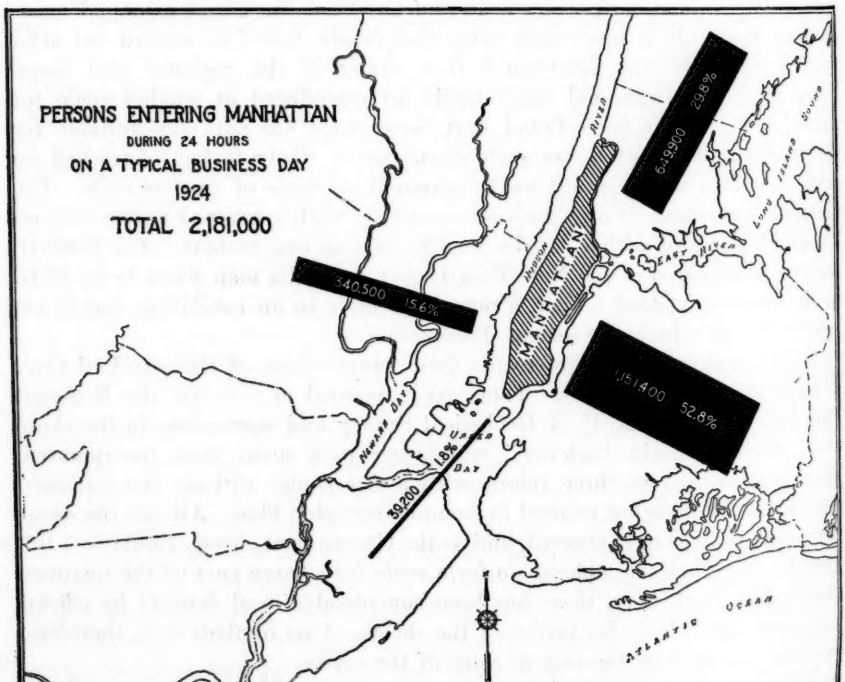

FIG. 1.—STUDY OF PASSENGER MOVEMENT.

The second map was a composite of all proposals for communication facilities of regional importance (as shown, for example, in Figs. 1 and 2) which seemed worthy of serious consideration, including those put forward by other

bodies and individuals and developed by the Regional Plan Staff. This was found invaluable as a basis of discussion of the relative merits of highway, railroad, transit, and waterway projects.

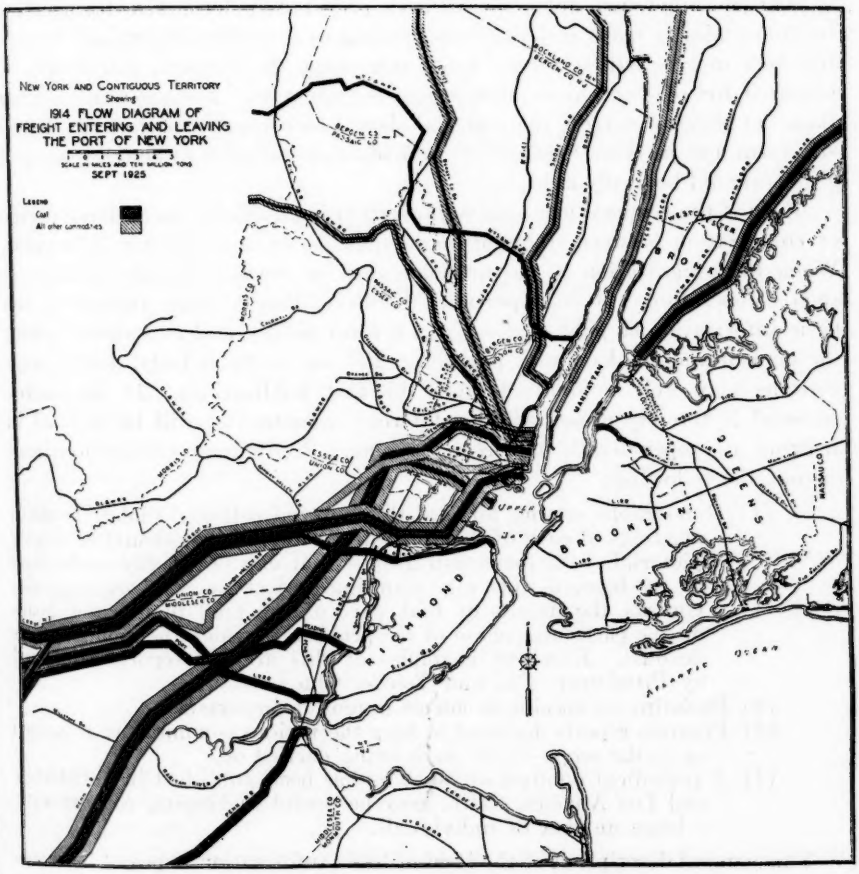

FIG. 2.—STUDY OF FREIGHT MOVEMENT.

The third map has been called the draft regional plan and is a first attempt at co-ordinating all proposals for land uses, communication facilities, water-front and port development, sanitation, etc. It is necessarily complicated but it has been found essential to present all such proposals on the same map so that they can be properly co-ordinated and their interdependence understood. It can be considered as the final major step in the accumulation and presentation of basic information. Conferences and discussions will surely lead to many changes in such a map, but if the work to this point has been carefully and thoroughly done the main elements of the regional plan should need only minor changes.

An important consideration in collecting the large amount of material required for a regional plan is the form in which it will be prepared for

office use. It is easy to lose a large number of valuable data through filing. It has been very helpful to prepare photostat copies of uniform size of all important maps and bind them in logical sequence. When a particular study is nearing completion, folders have been prepared in letter-size form, which include copies of maps and diagrams relating to a specific subject and descriptive text in typewritten form. From this shape the material can readily be prepared for publication in monographs or bulletins. For the studies other than the physical survey, the material should be prepared in the same letter-size form, but in these cases it will be mainly descriptive, although maps and graphs should be freely used.

Even if the regional planning organization is an official one, a large degree of co-operation between different municipalities is essential for its success. When the organization is an unofficial one, its success depends entirely on such co-operation. It is important, therefore, that a large amount of the basic information be published early in a form readily understandable to both the laymen and the local city planners. For an unofficial body, definite suggestions may well be omitted from the first publications. If the survey material is forcibly presented, the different communities will be assisted in arriving at proper details of a regional plan. Preliminary publications may include the following:

(1) Monographs setting forward the material collected under the main headings of each of the different surveys. These should be largely pictorial, with maps and graphs that can be readily understood by the layman. (In city planning projects such reports may also contain the details of that part of the final plan which bears on the particular phase of the problem to which the monograph is devoted. Excellent examples of this are the reports published by Pittsburgh, Pa., and Toledo, Ohio).
(2) Bulletins on specific problems urgent in importance.
(3) Progress reports designed to keep the various communities informed as to the scope of the work being carried on.
(4) A periodical publication, such as has been published in Pittsburgh and Los Angeles, Calif., may be useful in keeping contact with a large number of individuals.

This general description of the kind of basic information required indicates that a regional plan is complicated. In addition to all such information as can be put down in written or map form, there is also required an intimate personal knowledge of the region, specialized technical ability along various lines, and a wide general knowledge of city and regional planning principles. These qualifications can seldom be combined in a single individual and regional planning is, therefore, essentially a group project. The eventual plan must be the evolution of a staff working in close co-operation and aided by the best expert advice available. It is necessarily a compromise between conflicting proposals and ambitions of the various communities affected. Although in its final form it may not seem best for an individual community, it probably will prove to be so, as only a well-rounded and balanced plan is practicable.

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PAPERS AND DISCUSSIONS

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HOUSING AND THE REGIONAL PLAN*

By John Ihlder,† Esq.

The great American novel, the coming of which we used to discuss with such animation, has not been written and probably never will be, for a novel must have location and must picture, with fidelity to detail, individual characters as well as groups of characters. This necessary detail inevitably makes it sectional, provincial, differentiates its people, and its atmosphere from the people and the atmosphere pictured in another novel equally good, equally true, the scene of which is laid in another part of the country, or which deals with a different group of people. These differences will cause Americans of other sections to disclaim both books as representative of America as they know it, although to foreigners, both books may seem typically American. We, close at hand, see most clearly the differences of detail, accept subconsciously the likenesses; they, farther away, miss the detail and see only the broad likenesses.

Thus, it must be with regional planning and the housing that is to develop as regional planning becomes accepted practice. The broad likeness that will be characteristic of regional plans and the housing for which they provide will lie in acceptance of the proposition that they shall provide adequately, even generously, in terms of open spaces, of sanitary equipment, of "modern" conveniences; that they shall be based on the well-known but, fortunately, never clearly defined "American standard of living"; fortunately, because this standard is ever changing, ever rising. Other nations may accept present standards, seeking merely to modify them so that they may be tolerable; other nations may calculate closely on economies which they believe the hard conditions of their life force upon them—definitely discard, for example,

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water-borne sewage and a sewerage system, not only because of cost of installation and operation, but also because of loss of fertilizing content which they believe they must have for their farms. Americans, however, who are coming to reckon farm productivity in terms of bushels per man while other nations reckon it in terms of bushels per acre, will base their plans on the health, efficiency, and more abundant living of the population rather than on the amount of money not spent for a sewerage system or an imported or manufactured fertilizer. Water carriage of sewage may go into the discard, but not until a better method of safeguarding human well-being has been found.

Inside such broad American characterization, however, regional plans will doubtless take on many differentiating characteristics due to sectional habits, traditions, resources, and climate. Consequently, in a paper like this, dealing with the subject for the whole Nation, one must paint in broad strokes, describe objectives in general terms that are subject to infinite modification in their detailed application, give approximations rather than exact measurements.

To engineers this may seem an indication of loose thinking, but knowledge of the fact that a 7½-ft. ceiling height may be adequate in Maine, whereas a 14-ft. ceiling height may be good economy in Southern Florida, gives the writer courage to persist.

A house is not a commodity of uniform sizes and character. Its variations are infinite, although they all fall into fairly clearly defined classifications. The use of each of these classifications will be affected by the regional plan if that plan proves effective in guiding metropolitan or regional development. Consequently, an outline of a regional plan from the housing point of view is necessary, if subsequently housing is to be fitted into it understandably.

Regional Plan Outline

The metropolitan region may roughly be defined as the area within commuting distance of the central or mother city. It may be an area about one or two cities of small or medium size, as well as that about a large city. At present, it may contain only one or two units of really urban character, the remainder of the region being residential suburban. If it does not have other urban units now, it will acquire them later as the population increases and neighborhood business districts develop. Moreover, in this day, when every chamber of commerce is seeking factories, and when factory managements are thinking in terms of industrial distribution, it is almost inevitable that the metropolitan region outside the mother city will develop industrial districts. Consequently, provision must be made in the regional plan for both industrial areas and commercial areas, in addition to residential areas. These, normally, will grow into separate towns, and, unless preventive measures are taken, they ultimately will merge into one great city, as Philadelphia, Pa., and its satellites have done.

Philadelphia has developed much as the regional planner would have a city develop, except that adequate provision was not made for traffic and

that the separate communities were not kept separate by adequate intervening open spaces. The original City of Philadelphia with its mill satellites, Kensington and Manayunk; its factory satellites, Nicetown and Tacony; its residential satellites, Germantown and Chestnut Hill, each separated from the others by open areas, the brook valleys preserved instead of being filled to make uncertain sites for the foundations of buildings, would have been a more pleasing city than it is, with its interminable streets filled with monotonous rows of houses. By the same token, the Philadelphia of to-day is a more pleasing city, with such access as it has to the open country, than it will be if its present streets stretch out and out until they join those of Chester and the other satellites that have succeeded to the opportunities once offered to Kensington and Germantown.

How Urban Units Are to Be Separated

The metropolitan region of the future, therefore, will be planned to contain a number of distinct urban communities which will be enabled to preserve their individuality by surrounding open areas. In part, these will consist of parks, some formal, such as Fairmount Park which to-day separates Philadelphia from the main-line towns, and some natural parks or forest reserves, such as those now being acquired by Boston, Mass., Chicago, Ill., and other cities. Park land alone, however, will not be enough. So much open space is required for the proper ventilation of cities that a considerable part of it must be devoted to productive uses that will pay its way. Cities like New York, N. Y., and its New Jersey neighbors, and the great congeries of cities that are growing up about San Francisco Bay have been blessed, largely against their wills, by open water spaces that seem to some persons far larger than are required—just as in the past with some cities, it was thought that certain streets were too wide, only to find to-day that they are not wide enough. It would be difficult to over-estimate the economic value of the breezes from the water that blow through the streets of New York and the Bay cities.

Awakening to the advantages which Nature forced on some fortunate cities, we shall provide adequately for the new kind of harbor that is coming with the aeroplane. We doubtless shall, under stress of necessity, calculate carefully how small an air field may be, how high the surrounding buildings may be permitted to rise, in order to bring the air harbor as far in town as possible so as to minimize change in existing values. As the railroad induced river towns to turn their backs on the levees so the aeroplane may induce them to face in a new direction, and those towns which make the most adequate provision are likely to reap a benefit.

Air harbors like water harbors, however, will prove inadequate to the purpose, and other uses, such as truck gardening and farming, will be found for these open areas.

How Urban Units Are to Be Connected

Isolation, however, is not an objective in the regional plan. Each of the urban units in the region must be readily accessible from the others, more accessible than the upper east side of New York from Riverside Drive. Water

routes, rail routes, air routes, and main or arterial highways will be carefully planned so that each center in the region may be easily and quickly reached from every other center. Remembering that by far the greatest daily travel of an urban population is between home and place of employment and that in such cities as New York or Chicago, with their overgrown central business districts, where vocations that have no direct connection are jumbled together, the working population must pass twice daily thousands of buildings that have no possible part in their lives, the waste of time, effort, and money becomes obvious. Although like businesses instinctively tend to group themselves, and thus simplify the transport problem of the workers after arrival for the day's work, these employees still waste much time every morning and evening in passing the buildings of other groups and the dwellings that house the employees of these other groups. The regional plan will reduce this waste by segregating vocational groups more effectively and will bring the homes and workshops of each group closer together. The need of occasional intercourse between representatives of different groups will be met by express routes, rail and highway—perhaps in the near future, air—between the urban centers in the region.

The better segregation of vocations, which can be secured by intelligent regional planning and zoning instead of depending on the blind instinct that has guided in the past, will bring the home and the work closer together and in smaller urban units. As a result not only will time of travel—to-day worse than wasted because of crowded cars that sap the rider's strength and vitality—be reduced, but the character of the dwelling may be improved because more space will be available.

Transit Routes as District Boundaries

The transit routes that tie the urban centers of the region together will largely form the boundaries of residential districts. Along parts of these routes, especially at the intersections of arterial highways, will be minor or neighborhood business districts, containing neighborhood stores, banks, moving picture theatres, etc. Within the space they bound will be a residential area large enough to support one or more schools, playgrounds, small neighborhood parks, a branch library, a community center, churches, so that children at least will have small occasion to cross the busy main traffic streets.

These main traffic streets or arterial highways are one of the most interesting of the problems that confront regional planners. Their primary function is to carry traffic from center to center within the region or to more distant destinations. How they should be designed, what width should be secured for rights of way to provide against future needs, whether they should contain facilities for rail as well as for road vehicles, are questions outside the scope of this paper. It is necessary, however, to point out that they will carry not only a heavy volumn of traffic, in some cases a constant stream of traffic by day and a considerable and increasing amount of traffic by night, but that they will carry heavy vehicles. The increasing weight of trucks and buses has become a matter of public concern. Admitting, what seems to be the fact, that while the large truck and bus make for economy of operation for their own-

ers, they at present cause great expense to the community as a whole and to property owners along many of their routes where they are permitted to range at will. Street paving that would carry passenger cars for years goes to pieces quickly under their pounding which also cracks the walls of dwelling houses, and which, together with their noise, seriously depreciates the value of whole residence districts. The main arterial highways apparently must be designed and built to carry vehicles that will some day be excluded from residential streets.

The old theory that every street-car street was potentially a business street, was a poor one and was based on inadequate experience. Inadequate as the basis always was, it is becoming every day less adequate. Not only are merchants realizing that the string business district can not compare with the compact district; not only are buses that operate on parallel streets applying the same argument to those parallel streets, but the public is learning that business can never occupy all the frontage on street-car streets. Recent studies have indicated that, outside the principal down-town shopping district, not more than 5% of an area will be occupied by business.

If this is borne out by further studies, it seems that the frontage on the main arterial highways of the region must in large part be devoted to other than business uses. Attempts to develop such frontages to residential use are not proving permanently successful, even for multi-family houses. The inhabitants of multi-family houses object less to noise and movement than those of one-family houses, but even they are beginning to find that noise and movement can be increased to such a degree as to become intolerable. Thus, although the non-business frontage of these arteries may be zoned for multi-family house occupancy, even such dwellings will have to be given protection if they are not to be blighted. The suggestion the writer would offer is that in addition to the ample right of way for traffic purposes, the community shall take possession of strips on either side of the traffic artery. These strips should be parked, and on each side of them should be a minor street serving the dwellings. The dwellings should then be set back, to provide for two screens of planting between them and the traffic highway. The park strip might be so wide that when business expansion can utilize it in part it will provide suitable sites.

This may seem extravagant but it will prove less costly than the slow and spotty development of property abutting directly upon a main traffic highway and the inevitable depreciation of such dwellings as may be there erected.

City Facilities and Neighborhood Facilities

Housing cannot be considered adequately apart from facilities offered by the city or community and their accessibility. To the wage earner distance from place of employment translated into terms of money for means of transit and time or effort is of greater importance than to those whose means are greater and hours of work shorter. To every one, however, the waste involved in needless daily travel is an economic loss of moment unless it can be transformed into a strength-giving, or health-giving, or inspiration-giving

factor. The well-to-do man who drives from a pleasant home to an office near a parkway, may well find this interval an asset, but one who must travel morning and evening in a closely packed subway train is almost sure to find it a distinct liability.

The best means of transit yet invented is that one, reminiscent of days before the horseless carriage, known as "shank's mare." For the normal man a half hour's walk, especially if it can be along a pleasant route, is a good prelude to a day of sedentary or indoor labor. Next, perhaps, comes the bicycle, for this too involves exercise. Then, come vehicles that give their occupants the benefit of sun and air, and last comes underground transit. As the old-fashioned, outside, iron fire escape, now happily disappearing, was a confession of failure to construct buildings properly, so the subway is a confession of failure to construct cities properly. There is room enough in the United States for all to live above ground except the miners. Such failures, forcing people to patronize crowded or sub-surface vehicles, have a direct effect on housing by compelling a choice of two evils—these means of transit, or an inferior type of dwelling nearer to the place of employment.

Although access to place of employment is usually the chief consideration, access to schools and to places of recreation, are of some importance. These merge into facilities offered by the neighborhood which is the area that, ideally, lies between main traffic streets and all parts of which are within easy walking distance of home even when home is part of a district of widely spaced, garden-surrounded one-family houses. Here will be the grade schools and the playgrounds for small children, perhaps a High School, or at least a Junior "High," for the older children, a library, or a community center, and a neighborhood playground for adults, and churches—set in grounds large enough to permit their expansion without violating the area provisions of the zoning code. Most congregations seem to be pessimistic of their future, yet instances are known of churches having had occasion to expand and their officers had no hesitation in asking that a rule made for the benefit of the community and the protection of their neighbors should be suspended in order that they might escape the penalty of lack of foresight or lack of faith.

Within the region and its units, thus broadly outlined, the problem is to fit housing so as to give the people of urban America the greatest possible facilities for living abundantly.

A Few Assumptions

In order to summarize the subject, several assumptions will be given, as follows:

1.—It is desirable to decrease the speculative elements in housing and to increase the investment element until the first has been reduced approximately to the vanishing point and the latter has become controlling. This means that values should be stabilized. Stabilization of house values is dependent on stabilizing the character of the neighborhood and in this stabilization regional planning, supplemented by zoning, is essential.

Conceding the great part that speculation has played in producing the dwellings of America, it must also be admitted that it has played as great a part in destroying them, in blighting whole sections of cities, in promoting rapid transition that lowers housing values so quickly that the investor has withdrawn from a large part of the market, leaving it to the "home owner" whose controlling motive is sentiment, in deteriorating construction until the buildings erected to-day have a much shorter expectation of life and a far higher expectation of repair and maintenance costs than those of our fathers and grandfathers, and, most important, in leading to the acceptance of dwellings inferior in type. From the one-family house it has led to the multi-family house and has gradually squeezed and cramped this lower form of dwelling until a large and increasing part of the urban population lives in one, two, and three-room apartments. These multi-family dwellings are popularly supposed to have investment value, but experience is showing that the rapid obsolescence of a multi-family house often prevents its being rated as a high-class investment. From the social point of view its destructive effect on family life is a matter of concern.

2.—The preservation of the family—meaning parents and children—is essential. The population of voting age may be everything that candidates for public office tell its members they are, but its life and its work would lose significance if there were no children to carry on. With the children lies the whole future. Consequently, children are of first importance. As the house is the shell in which the family functions, as it exerts a constant influence in moulding and shaping the family, even in determining whether or not there shall be children, the question of housing should be approached from the point of view of the well-being of children.

3.—The one-family house with generous open spaces about it is the best house for a child. Consequently, every effort should be made to promote the erection, to protect the continued existence of such houses.

4.—While the one-family house is the most important type of dwelling, there is a legitimate demand for other types, ranging from the two-family house through the so-called multi-family house and the apartment hotel to the hotel. This demand must be met, but because these socially inferior types of dwellings with their possibilities of land overcrowding and cramped living quarters can under-live the one-family house and drive it out, just as Oriental labor can under-live and drive out white labor, it must be restricted to certain specified sections of the community and must be strictly regulated so that it will provide the essentials of wholesome living—light, air, room space, sanitation—for its inhabitants.

Classification of Urban Population

In urban communities there are several easily recognized groups for each of which housing provision should be made. First and much the most important, because with them lies the future, are the families with children or expectation of children. They call for first consideration. They them-

selves may be divided into two classes. First are those who are fairly permanently located in the community, whose interests and fortunes have been and will be bound up with those of their neighbors and fellow citizens. They are the most valuable element for they have developed or can develop a lively sense of community responsibility. Less valuable, but more in need of assistance, is the second class, composed of families which, because of the nature of the bread winner's work or because of the temperament of father or mother, frequently move from city to city. They range from high salaried officials of large corporations, army and navy officers, professional men and women—civil engineers, social workers—to the automobile tramps who have become an interesting and puzzling phenomenon of modern life, whose younger children have never known a more stable home than the "flivver" and whose importunities are increasing the burden carried by charity organizations. Even those among this second class who are best placed economically have a difficult problem in providing homes for their children in these days when the choice lies between buying a proper house or renting an apartment.

In the first group it will be noted that families with "expectation of children" have been included as well as those with children. There is a great deal of talk about giving the young married couple a shelter that will just fit their present needs, assuming that when the expected happens they will move from their furnished two-room flat to that idyllic vine-covered cottage where love traditionally abides. Considered as a matter of pure economics, there is much to be said for all this, but while sound economics should be the foundation of living, pure economics is a sterile soil which will not produce an adequate crop of babies. Marriage, the family, is an adventure. Reason it out too coldly, balance economic items too carefully, and the young couple will grow to middle age, still living in their apartment, still thinking first of their own safety, their own comforts. Then the Nation may well ask why it was taxed to provide for their schooling, to protect their health, when they have been unwilling to pass on the heritage they received. The first home of the young couple should be at least a promise of its future home, should have in it the room, the play-yard that every day ask when the expected is to arrive. This may be economic waste, but the greater part of the joy of living consists of what cannot be strictly justified on the score of pure economics; it may be beyond the means of many young married couples, but it is an objective to be approximated as closely as we—and they—can.

Next to be considered, because they have not shirked but have rendered their service to society, is the group composed of those who have reared their children and sent them out into the world. The home that sheltered them when all the family were together may now be too large, too much of a burden. Many will continue to maintain it because of sentiment, but others will desire and should have a more convenient shelter. Their problem is not met by old folks' homes, however those may be disguised by luxury. Perhaps the nearest approach to a solution is the occasional multi-family dwelling where through some happy circumstance of management, tenant leadership, or a common

dining room, the inhabitants mingle for a time in the evening, and the older people have opportunity to maintain some daily contact with younger people.

Then come the unattached individuals who form the tragedy of civilization, often not recognized by its victims until they reach middle age. Their variety is so great, class merges into class so imperceptibly that it is difficult to classify them definitely. They range all the way from the well-to-do bachelor who lives at his club and thus has the casual social intercourse with his fellows that fills so many of the odd moments of contented living, the lack of which reduces living to periods of conscious effort interspersed with periods of loneliness; from the two spinsters who have joined forces to fight off loneliness and who live together in a little apartment, through those forlorn ones who inhabit boarding houses—a form of housing now apparently on the decrease—hotels and rooming houses—a form of housing now apparently on the increase. This great army is recruited from the youth of the land who venture forth in search of fortune. Its veterans are those who fail to make a family harbor. The problem of the unattached, whether it be the well-known "homeless man" who patronizes municipal lodging houses, or the wage-earning woman, whether it be the raw recruit or the veteran left-over, is one that has not yet begun to be solved in spite of the voluminous literature dealing with fragments of it—perhaps because this literature does deal with fragments only instead of with the problem as a whole.

Classification of Houses

For the population that is to be housed in a carefully planned region there is choice among the following types of dwellings:

One-family houses (one family occupying the whole house from cellar to roof.): Detached; semi-detached; group; or, row.

Two-family houses (one family above the other) subdivided as previously described.

Multi-family houses (ranging from the building in which every apartment is equipped for housekeeping, through that where a common dining room supplements or supersedes the housewife's efforts, to the hotel where cooking in an apartment is strictly forbidden.) Detached; group; or row.

Somewhat apart from any of those mentioned are the boarding house and the lodging or rooming house. These are not a distinct type of housing, but are merely the result of opportunist attempts to utilize the waste resultant from lack of city planning and a housing policy in the past. When the day comes that there are no more blighted districts, no more cast-off dwellings, the boarding house and rooming house, as they are known to-day, will disappear, their places being taken by houses designed for the purpose. Instead of a shame-faced dilapidation, recalling better days, they will evidence the self-respect of those who accomplish what they intend to do.

With this classification of houses; with a clear understanding of the function of each class; with a regional plan, zoning regulation, and an intelligent distribution of centers of employment so that there will be ready access from

home to shop; with a stabilization of the character of neighborhoods and, consequently, of house values, and, not least important, a clearer recognition of the value of space, both inside and outside the house, but part of the same domain so that alterations and improvements may be made, it will be comparatively easy to develop a housing policy designed to serve adequately the needs of the population.

Each individual has conflicting desires among which he must choose. Each might prefer to live in the White House, not because of an ambition to be President, but because the house has the desirable characteristics of a family dwelling; it is spacious and set in a very pleasant yard large enough for children's play and even contains a tennis court, and, in addition, it is most accessible from its tenant's office which is easily reached by his chief business associates, and, to add excellence to excellence, to satisfy the other head of the family, it is cheek by jowl with the principal shopping district, within a block or so of two theatres, and within easy reach of the others. In short, it would seem to be ideal. Certainly, as Lincoln is reported to have observed, most of its tenants desire to renew their leases.

Although this combination is provided for Presidents, most families have to make choices. If convenience of access of department stores is more important than home or children, an apartment near the center of one of the larger urban units or along one of the arterial highways is selected. If we no longer have children or expectation of them, a similar choice may be made, although some recent sub-divisions give reason to hope that it will be possible to find a small house of five or six rooms, attractively designed and located in a pleasant neighborhood, thus doing away with the present hard choice between an eight to ten-room house and a five-room apartment. If there are growing children a one-family dwelling will be chosen, a little less accessible from the centers of work and amusement, but compensating for this by giving neighbors who have the same chief interest and who have a greater tendency to stay put long enough for the development of family acquaintance, parents with neighbors' children, as well as the horizontal acquaintance of apartment-house populations which tends to follow the line of age groups.

The greater stabilization of the character of neighborhoods will encourage investment in rental housing by increasing the life expectancy of the individual house. It will lead to the wider use in one-family house districts of services now characteristic of multi-family houses. Stabilization which reduces the speculative factor in real estate, which turns attention to permanent investment values, should result in creating again the estate or company that owns or manages a considerable number of one-family houses, for these depreciate, become obsolescent less rapidly, cost less to maintain and, provided they have open space about them, are more readily kept in step with "modern improvement," than are multi-family dwellings. Some of the finest dwellings are one-family houses built 50 or 100 years ago. Some of the old one-family house districts after a period of decadence have come back strongly. Few multi-family dwellings have maintained the standing of their

youth until reaching their majority, and none, so far as the writer knows, has ever come back after it once lost prestige.

Probably this has been due chiefly to misplacing. They have themselves spoiled many neighborhoods and, in time, have suffered from the deterioration they caused; or they have been injured by the invasion of business. In a planned and zoned region these causes should be removed. There will obviously be space enough so that the temptation to land-overcrowding will be reduced. There will be system and order so that each type of dwelling will have that place best fitted to its purpose. Zoning does not imply a series of girdles about an urban center, but it does imply an arrangement in relation to traffic and traffic facilities. The regional plan will guide the development of traffic facilities, types of dwellings will be placed in accordance with their need of these facilities, and zoning regulations will prevent the placing of an inferior type in a district where it does not belong.

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CULTURAL OPPORTUNITIES IN REGIONAL PLANNING*

BY ANDREW WRIGHT CRAWFORD,† ESQ.

On his return from his last tour through the Southern and Western States, Otis Skinner related that he had done numerous one or two-night stands in typical "Main Street" towns, with all the stale, flat dullness that one associates with that description. In nearly every case, however, some one asked him to play on the golf course, which he generally did; and he always found the delightful, charming loveliness, with all the beauty and abiding cheerfulness of Nature, which the well-kept swards and the long green vistas, required by the game, produce at every turn.

The introduction of golf has brought a cultural opportunity to the dullest, drabbest town that boasts a course. Golf courses are the anti-toxin of "Main Street". Doctors state that golf is prolonging the lives of many men, but, important as that service is, it seems surpassed by this observation of Mr. Skinner. Many individuals will echo an agreement, and give parallel citations of their own experiences. "Bangor" used to bring the recollection of terrible meals in a wretched station in a hopeless town; but now the name conjures the vivid picture of a river vista bordered by the tapis vert of the golf course, dotted with glorious New England elms.

Culture is assisted much by education, much by travel, much by reading, but it depends in large measure upon physical adjuncts. It is possible to conceive of a cultured man living in a town with no schools, no libraries, no playgrounds, no parks, no golf courses. But while one individual, like a "sport" in horticulture, might arrive and stay in such a place, it is not reasonably possible to conceive of such a town becoming a community of cultured men and women. The opportunity has to be present. Schools for the children, with the play space where they learn so much, libraries, churches, and community centers must be provided. Consider the charm of an ancient street of a

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New England town with great elms arching over it in cathedral forms. The mere thought suggests refinement. Cut every tree down and the suggestion is obliterated.

If the regional plan gives adequate locations for libraries, for common and high schools, for colleges, for universities—all on the educational side; if locations are plotted for playgrounds—preferably to be recreational playgrounds—although in crowded centers they must be largely pedagogical playgrounds; if the opportunity to get back to Nature for city people is provided through country parks in sufficient number to make it a frequent experience; if the regional plan gives sufficient examples of the plotting of street trees with as much care and particularity as the plotting of street conduits (be it noted that in some foreign countries the engineering plan always shows the locations of trees); if these things are done, then the regional plan will be of distinct service in helping to create the physical surroundings which are an important aid in the production of a cultivated community.

This is not all. If public structures, or structures extending over public property, like bridges of all types, signs, lighting fixtures, etc., are well designed, they will be at least as cheap as ugly structures. They will add to or detract from values of the neighborhood in exact proportion to their beauty. An ugly bridge seldom lasts out its life. The community gets rid of it before its expectation of life is run. An engineer who does not point that out to his client, when the latter wants him to design an ugly bridge (but which he calls "merely utilitarian"), fails in his duty to that client.

An investment in ugliness is as extravagant an investment as the directors of a corporation, the council of a city, or any group may make. If Washington, Paris, and Pasadena, show the money-making power of municipal attractiveness, how completely do Hoboken, Weehawken, and Manayunk demonstrate the sheer cost of ugliness. Effective regional planners, therefore, will see to it that over each and every region into which the regional plan reaches, some authority is recommended to be set up, like the State Art Commissioners or Art Juries—of which there are now about thirty in the United States—which will consist of men who will have the backbone to say to politicians seeking favors for important voters, that no ugly thing will be permitted to depreciate the values, financial and cultural, of their city, suburb, or country. An ugly bridge decreases all values around it, especially in suburban sections, and an attractive one increases that value.

In Philadelphia, the tradition of attractiveness in suburban development has become so strong that its suburbs have become world famous. Nevertheless, much could be done in this region to improve the architecture of the smaller houses. In one or two foreign countries, "Bureaus of Building Advice" have been created, the function of which parallels for private owners (who purpose to build on private property) those of the public art commissions as to public buildings. Plans are submitted to the "Bureau of Building Advice" and it advises the owners as to the effect of the appearance of their buildings on the values and worth of the community. The owners do not have to accept the advice; but they do not care to have the advertisement that their

buildings have been condemned by the "Bureau of Building Advice" and, consequently, they do accept the advice in many cases. It would be well, therefore, for the regional planning central group to recommend and then to secure the creation of such "Bureaus of Building Advice" in their community.

Fifty years ago individual parks were regarded as luxuries. Thirty years ago they were accepted as necessities, but parkways to link them into an organic whole were regarded as luxuries. To-day, parkways are regarded as necessities, and some automobilists would have us create super-parkways to give them super-opportunities for killing others as well as themselves.

It is assumed that the creation of an adequate park system, including the reservation of creek valleys, the preservation of places of natural beauty or historic interest, the acquisition of certain high plateaus in the region for public use and public enjoyment, will be accepted for what they are, as fundamental portions of the regional plan.

It is important, however, to refer to two points—one of which is obvious and more or less becoming the practice, but the other of which, curiously, has been lost to sight. Parkways should be regarded as traffic thoroughfares. Opportunities for the operators of motor trucks, as well as of pleasure vehicles, to drive, in the course of their daily employment, or on their way to or from work, through attractive thoroughfares, is one which is somewhat recognized, but not as completely as it should be. The idea that a park or parkway is somewhat the old-fashioned parlor—only to be used on Sunday when the parson comes for dinner after church—is one that should be completely discarded—the idea, not the parson. Motor-truck routes and motor touring-car routes should be devised for their every-day attractiveness and for every hour of the day. This means occasionally a slightly additional expense for width. It means hardly anything in the way of the development expense of the property; perhaps it really means a reduction of cost, instead of an increase, because property fronting on a parkway is more valuable than one that does not.

The second point that it is desired to emphasize in this respect is the one that has been forgotten. A practice has grown that is in direct contrast with the theory with which modern city planning in America was begun about twenty years ago. It was the basic argument for city planning then that one improvement to a city frequently was plotted, or designed, or conceived solely with reference to that improvement and without reference to the program of the city as a whole and the proposed order for other adjacent improvements. City planners insisted that this was wrong and that the entire problem should be considered together. Then zoning came along and many city planners lost sight of their argument for comprehensiveness. Zoning systems, plotted without regard to park systems, are numerous. Zoners think that if they zone property in a certain way because it fronts on a park, that they have sufficiently recognized the park system.

One has yet to see a zoning plan in which zoning for use is made complete. If property is to be used for residences, business, or manufacturing, the zoning plan shows it; but if the property should be used for parks the zoning plan by

itself does not show that use. Obviously, a zoning plan should suggest areas for public use in the form of parks or playgrounds, just as it should zone areas for private use in the form of single-family and multiple-family houses, commerce, or industry. Zoners have failed adequately to see the opportunity for living up to the principle for which the city planners began twenty years ago—of preparing the plan in its entirety.

In addition, however, an opportunity for parks in connection with zoning, has certainly not been emphasized. There is no reason why zoners should not frankly divide a commercial or a transportation industry from a residential district by the interposition of a public park. True, it may not be so acquired, but it should be submitted as a recommendation. In Philadelphia, there is such a natural division. The Midvale Steel Works lies in a valley and a suburban section of Southern Germantown lies just over the crest of the hill. The hill and the slope are occupied by Fernhill Park—53 acres in extent. Why should not this principle be frequently applied in zoning?

The opportunity is especially available in connection with regional planning. When a region is planned the opportunity for placing elongated, wide, or narrow parks as a boundary, with residences on one side and heavy or light industry or business on the other, is so obvious and the opportunity is so great that this paper will be considered worth while if it conveys this thought, plus Otis Skinner's observation of the cultural power of golf courses.

Sixty to seventy-five years ago, when American cities first began to acquire such parks as Fairmount Park, Central Park, Franklin Park, and Druid Hill Park—in Philadelphia, New York, Boston, and Baltimore, respectively—it was thought that just one big park was all that was necessary. When the writer became Secretary of the City Parks Association, in 1900, there was put into his hands a pamphlet by President Elliot, of Harvard University, on the question of how to get the people to the parks. It has been answered by taking the parks to the people in the form of small parks and the park system, by establishing, as it were, a central park and branch parks.

The same psychological change has occurred in connection with libraries. The great Public Library in Boston—great because of its beauty, not because of its size—was just in course of completion a little more than a third of a century ago. In that short space of time the need for taking the library to the people has been recognized, and every large city has built many branch libraries. Numerous American cities (Philadelphia among them) have recently completed or are completing great central art museums. They are absolutely important to the cultural development of the city; but equally important will be branch art museums throughout the region. The recognition of this psychology is just at the beginning. We had to take parks to the people. We had to take libraries to the people. We equally have to take art museums to the people. Only within the last year has New York City opened its first Branch Museum, at 196th Street. The regional plan must, of necessity, if it is to be adequate to its cultural opportunities, plot the locations of a sufficient number of branch art museums to take pictures to the people as effectively as the branch libraries have taken books to the people.

It is suggested that the next generation will create branch universities. Imagine the University of Pennsylvania with eight or ten branches in the region within 30 or 40 miles of Philadelphia. It is not such a great advance over what has already been done, educationally, as it appears. In 1890, there was just one High School in Philadelphia. To-day, there are eleven High Schools and twelve Junior High Schools. In other words, the Central High School—a City College—is supplemented by branch high schools, some of the teachers of which think their own branches are higher than the High School. In some respects their thought is true. It would be interesting, or at least advantageous, to have locations for branches of institutions of higher learning in outer regions suggested by the regional plan. The first regional plan that does so will be regarded in the future as prophetic.

What is the opportunity of civil engineers in presenting on paper plans directed to the securing of locations in the region for the creation of cultural opportunities in all these different and diverse ways? What are the qualifications of civil engineers for the job? Well, it may be stated, in the first place, they understand construction; but only a few of them understand that construction is not design. Here is an illustration. If a plumber attaches three pipes to a main so that the water spouts out the other ends, he has not designed a fountain—he has constructed it; he has done the plumbing part. If an engineer builds a bridge merely sufficient to stand the traffic, he has not designed the bridge; he has only constructed it; he is only a plumber. If an engineer takes the trunk of a tree and makes kitchen chairs sufficiently strong to stand the weight of the cook, he has constructed the chair, but he has not designed it. An artist, taking another portion of that same tree trunk, will, by adding design to construction, produce a Chippendale chair worth $40 compared to the engineer's chair with a value of $4. Only those engineers who know design as distinct from construction should have anything to do with the regional plan. If that cuts out 95%, they can overcome the difficulty by studying design as distinct from construction. Again, such constructors can prevent other engineers from lacking what they lack by insisting that institutes of technology shall teach design to engineers, as well as how to calculate strength of materials, stresses, and strains.

Some civil engineers have felt justified in complaining of the injection of architects and landscape designers into city planning, because it seemed especially the field of civil engineers; but the civil engineers had the field entirely to themselves for the whole of the Nineteenth Century, and they have only themselves to blame that the public, weary at last of the drab, stupid, stale rectangularity of the checker-board cities which were all the civil engineers of a century produced, relegated them to the subordinate position of drawing the working plans for designs conceived by others. "By their fruits, ye shall know them." And the public has judged of civil engineers as city planners by the Nineteenth Century gridiron. The end is not yet. A graduate of civil engineering asked the writer whether, as the result of fifteen years direct contact with the supervision of plans for bridges, he thought architects would supersede engineers as designers of bridges. The answer was that it depends on

whether engineers learn to design—present trends indicate they would not—and that, with the astonishing advance in the determination of American cities to use Art as their daily servitor, the architects in another decade will have pushed the engineers down to the subordinate position of calculating stresses and strains for the plans of bridges designed by their architect-employers.

One had only to go to the Sesqui-Centennial in Philadelphia to see how Art has come into its own. If the Sesqui Art Museum and all it contained were entirely removed, the whole exhibition would remain a great art exhibit. Consider how the steel industry was most conspicuously represented. Jones and Laughlin, steel manufacturers, showed what? They showed what was essentially a notable work in sculpture. It could have been with advantage less literal; but the whole conception was fundamentally sculptural. That was the method of appeal to the public; that was the advertisement chosen by a great steel concern. In the housing of individual exhibits, every form of Grecian, Roman, and Byzantine architecture was used.

The mere engineer will become only a skilled workman unless he looks to design. What should those engineers, qualified by a knowledge of design, as well as of construction, for the job of regional planning—what should they study, what facts should they ascertain in order to be able to plan for the cultural development of an entire region? Chief traffic routes must be studied. Such mistakes, as that in New York, where Central Park blocks two vitally needed north and south routes, must be avoided. Opportunities along the traffic routes for the preservation of places of natural beauty and historic interest should be determined.

Obviously, the topography must be studied. A poor topography may require a somewhat larger area for a park system than a favorable one. The City of Philadelphia is fortunate. Such a deep narrow valley as the Wissahickon requires less area for the preservation of extraordinarily great natural charm, than a level terrain would have required. The percentage of the total area which should be taken for the park system necessarily will vary with the topography, with the character of the community so far as it can be predetermined by zoning, and with other elements. Ten per cent. is a principle which has stood the test of many years. Probably a good regional park system will approximate that. Larger playfields, covering 10 to 20 acres, should be plotted about 1 mile from each other, and the smaller playgrounds within 5 min. of each child's home. This, again, will vary in accordance with the expected intensity of residential development as forecast by the zoning system

In connection with playgrounds, and harking back to Otis Skinner, the writer would emphasize the importance of pay-as-you-play golf courses, and, in addition, a method for making privately owned golf courses permanent in their locations. Golf courses are open spaces. If completely permanent they would be open air parks, by which is meant parks which would be enjoyed by the public even if it could not walk over them. How can they be made as nearly permanent as possible?

It is submitted that if the golf club is willing to throw its course open to any member of the public who behaves himself and who pays the greens' fee, which averages from $1 to $3, that would be a sufficient justification, either for entirely exempting the golf club from taxes or for making the rate, say, about one-half the ordinary tax rate. In the City of Philadelphia there are three tax rates—the "city" rate, the "suburban" rate, and the "rural" rate. The suggestion is that taxes on golf courses, which are open to those who pay the greens' fee, should be either the rural rate or else that the taxes should be waived altogether. This would require legislation, but it would be a cheap means of increasing the permanent open spaces of a region. Within a journey of ½ or ¾ hour of Philadelphia there are more than sixty golf courses—which means there are sixty open spaces, averaging about 100 acres. If all these spaces could be made permanent by a stroke of the pen, it would be a blessing. Freedom from taxation would be a great incentive to a club to remain where it is. Otherwise, increases in values may not only make it desirable to move, but may make it imperative.

There is one other thought in this respect. If any golf club intends to sell its property, it ought to give some public authority about six months' opportunity to purchase it at a price equal to the highest price offered.

Zoning will also help to predetermine the natural location for school houses and libraries. If zoning, in its logical development, is finally sustained by the Courts, the needs of the region and of each section of the region in respect to thoroughfares, all sorts of public conveniences will be able to be forecasted, for fifteen or twenty years, with much greater accuracy than at present. Nevertheless, forecasts must be attempted through the familiar methods of studying the growth of communities by maps and otherwise.

In answer to the query as to the kind of investigation that should be made, in order to plan effectively for the cultural development of a community, so far as physical accomplishments can help toward that development, the items of calculation and of investigation are pretty much the same as in regard to the other elements of the regional plan. Growth of population, directions of that growth, topography, natural and artificial barriers, locations of railroad, rapid transit lines, traffic arteries, and secondary streets, all are to be included. It would be easy to take the ordinary list of divisions and sub-divisions of a schedule, and repeat, but to engineers it is unnecessary. Indeed, when one takes up one of the published summaries and sub-divisions presented for an ideal investigation, the effect is appalling. The investigator is apt to think that each particular head and sub-head must be investigated with equal meticulousness before he dare make a decision.

If to-day one could present a perfect plan for Metropolitan Philadelphia, twelve months later it would be not more than 97½% perfect. In four years, it might not be more than 90% perfect. Most big plans take at least ten years to realize, and a regional plan will take thirty or forty years. A perfect plan to-day may not be more than 60% of perfection by the end of thirty years. Every city plan and every regional plan that is under way to-day can be convicted of too much meticulousness in the information sought. The time spent

in investigating and getting minute details is so great and the value of those details is so negligible that a great deal of impatience with regional planning is justified.

It is necessary, therefore, to determine when all the really worth while investigation has been made, and all the information that is important has been obtained, but use much good sense as well as "horse" sense in determining that which is important. Hasty decisions are not advocated, but delayed decisions are strongly deprecated. These lists and tables of matters to be investigated have written over them on each page, in enormous letters so big that they are never seen, the words "Excuses for Delays". Refuse to emphasize the various points that might be investigated. The engineer can readily find such schedules, but when he finds them he should spend no more time than is absolutely necessary in answering each of them and should see whether genuine efficiency, not the meticulous kind so much preached and regrettably also practiced, does not dictate that many of them go uninvestigated. If a physician made every possible investigation that he might make in the case of each patient, his patient would probably die before he were a quarter finished.

It is important, too, to recollect that some of the modern gods may not prove immortal. Especially is this true of the god, Speed. To-day, speed automobilists have the public by the throat. All they have to do is to present an appeal against some kind of obstruction to infinite speed in order to have that obstruction wiped away. If a circular park at an intersection can be shown to delay automobilists 2 or 3 sec., weak-kneed city officials remove the circle. Recently, a Philadelphian took an educated Chinaman from the Philadelphia Sesqui-Centennial Exhibition to a charming suburban home, ten miles out, going by a beautiful route. On returning, he told his guest that they would come back by the less interesting Lincoln Highway, because it would save 10 min., whereupon the Chinaman said, "And what will you do with it?" The automobilist, abashed, was unable to reply. Propaganda for speed is in sad need of debunking.

A cultivated community may come accidentally, but it will come far more adequately and far more certainly if it is consciously planned. The ideals of one generation become the accepted realities of the next—sometimes sooner. In 1906 an intimate friend—the city editor of a great Philadelphia daily—referring to a project at which many were hammering away, said: "The Fairmount Parkway is a dream". In 1926 many used the dream of 1906 in coming to this Convention.

Engineers are practical men and because of that fact the writer wants to emphasize this thought—that dreams are the most practical forces in the world. Engineers can be mere technicians or they can be much more. Remember that mere dreamers have accomplished far more than mere engineers? Why not be both engineer and dreamer?

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FORECAST: THE REGIONAL COMMUNITY OF THE FUTURE*

By Thomas Adams,† Esq.

Introduction

The modern city is a product of science—applied to manufacture, to transportation, and to modes of living. It reveals in its conditions of growth that science may deflect the path of society and still do little or nothing to arrest the weaknesses that accompany the changes or movements inherent in human action. Yet the science that has brought about so much progress in the arts of industry and transportation, apparently could, if applied collectively on a wide community basis, make for better cities. It must be admitted, however, that whatever social progress may be attained, or expected, in this direction, by means of physical improvements, can only lessen and not remove defects in living and working conditions.

The complex dynamic forces that enter into the composition of the city may lead in the future along one of a thousand different avenues of growth. Who can calculate all the forces that might enter into one of these possibilities? Who can predict the periods and degrees of advancement and recoil, of action and re-action, of growth and decay, and, therefore, calculate their effect? Who can visualize how changes and innovations in individual and collective practice will operate to bring about new trends in growth; or in what measure and over what periods the government of the future will be influenced by principle or expediency? These questions indicate how rash it is to try to picture what the future regional community will be like. Yet, it is worth while to weigh and investigate future possibilities and opportunities in the light of knowledge of facts and tendencies of community

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growth; it is worth while to attempt to create some practical ideal and try to guide communities toward its achievement.

William James has stated, "If shown a certain way, a community may take it; if not, it will never find it."

The function of the city planner is to show the community the way that leads toward the healthiest and most efficient pattern of town development. To do so he must first have developed in his mind an ideal pattern which he conceives to be within practical reach; and, second, have worked out the ways along which it is possible and best for the community to travel toward its achievement. Lack of this ideal for the community, or of an intelligent means of reaching it, and not technical imperfections, lies at the foundation of many failures in city planning and zoning.

It is the writer's purpose to give a general conception of the regional community of the future, based on the belief that the growth of the science and art of city planning, aided by the play of economic forces, will influence the direction and form of urban growth toward a higher and more ideal civic pattern and more rational forms of development. Before presenting this general conception, some of the conditions and problems of the modern city will be discussed.

Some Conditions and Problems of Modern Cities

The modern city does not differ from cities of previous times in regard to the underlying character and causes of its defects; changes have rid it of some evils and brought others to take their place. The large city has always suffered, in different forms, from congestion of building and traffic, from poverty and bad housing, and from the inherent selfishness of man in dealing with private property. Since these are the results of human tendencies, that do not change with time and circumstance, they will always be present. The conditions and problems of New York, N. Y., and Philadelphia, Pa., differ in character and to some extent in degree from those of thirty or forty years ago. The automobile has not made congestion, but has created new forms of congestion; and it is still possible to move about the congested centers of the great cities with more expedition and more agreeably than in the days of horse-drawn vehicles. The high building has brought new kinds of evils, but is not so unhealthy in some respects as the unsanitary building facing the narrow streets of medieval cities. Science has relieved poverty by providing man with more abundance of food and clothing, but in the great city the poorer classes still suffer from low standards of housing accommodations and bad environment. The control of private property is effected by laws that lag far behind changes in growth. In spite of all the lessons of the past, man is too slow in adjusting himself to changes of conditions. This is what is meant by lack of foresight in dealing with the problems of the city.

The conditions and problems of the modern great city that are exceptional, are the results of the unprecedented size and rapidity of growth of urban aggregations. For instance:

(1) Evils of congestion that were bearable in small areas, where development proceeded slowly, have created almost intolerable conditions when extended over large rapidly growing urban areas.

(2) Separation of the home from the place of occupation of the worker, desirable in some respects in the smaller cities, has intensified congestion and caused serious economic waste in the large cities of to-day.

(3) Lack of space for movement, light, air, and recreation in the crowded central areas of cities was not a serious problem when the country fields lay within walking distance; but in the big city that spreads its buildings over 1000 sq. miles or more, it is found to be the chief defect in its growth.

(4) Co-ordination of municipal action in matters of public improvements and utilities was hardly necessary when cities were small, of comparatively slow growth, and widely scattered; to-day, every metropolitan region is suffering serious injury from lack of co-ordination between its constituent communities.

It is evident, therefore, that the demand for regional study and treatment of large cities is in large measure the result of peculiarities of modern urban growth. In approaching the study of regional problems, the unknown factors are so many and knowledge of past growth is so illusive as a basis for planning for the future that scientific investigation will be of only limited value. The investigations have to be reinforced in a special degree by reason and common sense, and the price of developing a new experience and a new science will have to be paid.

Important Trends in City Development

Tendencies in connection with transportation by rail and highway within regional areas are all toward facilitating quicker movement and more efficient distribution. These tendencies will continue to promote concentration of large populations in urban regions. This concentration has of course a great economic value, and as long as it is not permitted to lead to congestion, it need not be injurious to health, safety, or general welfare. The causes of congestion, as distinct from an efficient degree of concentration, lie in the lack of proper control of land development; the support that is given by law to abuses of property rights; the lack of proper standards of density and height of buildings as a basis for zoning; and the lack of harmony between physical planning and zoning regulations. The greatest menace to the modern city is that it will persist in avoiding the removal of these causes and simultaneously develop its facilities for transportation and promote new sources of energy. These facilities and forces will produce good or ill in proportion as the concentration they cause will be well or badly regulated by law and planning.

Perhaps the most hopeful fact in the metropolitan regions of America is the extent to which land is being acquired for parks. It may be many years before zoning of private property can be based on standards that will give cities ample space for freedom of movement and for light and air in buildings, and before it can be given a permanence that will make it really effective

in preventing congestion; it may be still longer before areas of land in these regions can be zoned for agriculture and horticulture and made integral parts of the open areas of cities; but the movement toward acquiring large areas of land for permanent open spaces for parks and parkways has gained an impetus that is full of promise as a means of giving future areas of urban expansion a large degree of lung space permanently dedicated to public use and ever present as a stimulus to better balanced development.

In the future, the architect, the engineer, and the social investigator will take a broader view of city problems. Zoning has already given a lead to a new architectural conception of city building, and as a force in town development it is still in its infancy. Scientific developments in relation to electric power, production of metals, improvement of machinery, and transportation; needs of great urban regions in regard to comprehensive systems of drainage, water supply, and adequate open spaces; these and other things will bring the engineer and the landscape architect face to face with great concrete problems of regional development, and make them more expert in the art of regulating city growth.

If nothing else forces action of governments and communities to plan comprehensively, compulsion will come from economic necessity. Great cities to-day are within sight of the end of the financial resources available to meet their needs, a condition that has been brought about by the wastes and inefficiencies incidental to economically unsound methods of development which permit unhealthy and unsafe conditions both of over-crowding in the central areas and of under-crowding in suburban and rural areas.

New York's Problems and Future Growth

Data regarding present and future conditions and problems in the New York Region will be referred to briefly.

The area of the City of New York is 298 sq. miles, but the area regarded as appropriate for regional planning around the Port of New York comprises 5 528 sq. miles. This area is in three States and has more than four hundred separate units of administration. This indicates a geographical extent and a complexity of physical and social conditions that sufficiently proves how extraordinarily different the modern city is from anything in the past.

In 1900 the population of the New York Region was 5 384 734; to-day, it is more than 9 000 000 and, in 1965, it is likely to be at least twice the latter figure.* About 58 000 dwellings are required annually to house the increasing population and between now and 1965 more than 2 000 000 dwellings are likely to be required. Whether or not much by way of reform of conditions affecting more than 2 000 000 buildings already erected, can be achieved, much can and should be achieved in influencing the proper development of an equal number in the next forty years.

The number of manufacturing plants in the New York Region grew from 19 416 in 1900 to 57 753 in 1920, or 197.5%; but the plants have become smaller on the average as the increase of employees is only about 100% in the

* Estimates based on past growth give 21 000 000 as the population in 1965.

same period. Railroad lines serving those industries have a mileage of 1 884; terminal facilities are inadequate for the existing need, and transit lines are ten years or more behind what is required for efficient handling of passengers and freight.

No prospective development of the water-front is likely to absorb more than a small percentage of its mileage for commercial purposes. At present, only 125 miles of 1 800 are so used, yet hundreds of miles are not being put to the best economic use because of false expectations regarding the potentialities of industrial growth.

The average density of population is 160 per acre in Manhattan, 30 per acre in the whole of New York City, including Manhattan, and 1 per acre in the remainder of the Region. In the city there is the comparatively ideal average of 7 houses to the acre, but in parts of Manhattan the density is as high as 150 families per acre. It is extraordinary how little variation there is in the average density of large cities and how congestion always seems to have its complement of vacant or sparsely developed land. To show that there need be no congestion with proper regulation of growth, 26 000 000 people can be provided for, at an average of between 20 and 30 to the acre, within a 30-mile radius of the center of the city. More even development must be secured if the decay that comes from the complementary processes of over and under crowding of city areas is to be prevented.

More than 2 800 000 people travel into and out of down-town Manhattan daily, but the crowding of this multitude into one small area will become less necessary as natural obstructions to horizontal growth are removed by improved transportation facilities. There is a growing tendency toward development of new neighborhood centers of manufacture and business. More belt-line communication is needed to give impetus to this movement. There is no true economic basis for the congestion of great cities like New York, and if it is continued the effect will be to strangle the pulse center of the community. A city has to avoid being stultified by its own wealth. Hilaire Belloc states that history proves that when cities reach their highest point of wealth they become congested, refuse to accept a remedy, and pass on to decay. He adds that:

"The main motive is a blunder in the science of economics. It is the idea that the destruction of a number of imginary economic values is an expenditure of real wealth. So far is this not being the case that there is no example in all history of a congested street system being reformed without the wealth of the city increasing after the change."

If, in future years, cities can be made to realize that great public improvements, if properly conceived, are good investments, the future can be viewed with optimism. In the Park Avenue District in New York the gain caused by the improvements within and around the Grand Central Terminal was 255% in land value and 932% in values of improvements in 20 years. The total gain was from $60 750 000 to $311 500 000, or 410 per cent.

Increases in land values are greatest in the centers and outlying suburbs. Great areas lie between where values are more or less stationary, and many areas are blighted. If people cannot live near their work they prefer to get

as near to the open country as possible. Tendencies toward decentralization are strong enough to overcome the difficulties inherent in making changes of location. The number of factories has decreased in Manhattan and increased in Brooklyn and Queens. Suburban increases of garages and theatres indicate the trend of outward movement. The rapid shifting of the population is doubling the number of children in suburban schools.

Heights of buildings even in New York have not reached the point of increasing average heights over older cities like Berlin, Paris, and central parts of London. The average height of buildings in Manhattan is only 4.8 stories, although there were 1 019 buildings between 10 and 19 stories in height in 1924 and 100 buildings of 20 stories and more, below 65th Street. Until more high buildings raise the average the effect of such buildings on traffic and business cannot be foretold.

High buildings have been the product of high land values and, in turn, high land values are causing high buildings. However, real estate owners and leading financial corporations now claim that the best business investments are low buildings, even in valuable locations. The following statements confirm the writer's views. Mr. W. Burke Harmon, of the National Real Estate Corporation, writes:

"All floors above the second floor can be largely eliminated as contributors toward the income of the building over a long period of years. The cost of upkeep of the upper stories and the interest on the investment represented thereby, plus taxes, would barely be met by the revenue derived from this space in normal times."

Mr. Richard M. Hurd, President of the Lawyers Mortgage Company, and an eminent authority on land values, writes:

"Low buildings in valuable locations pay a much higher percentage of income than high buildings. This is in line with my comments on office buildings, on which, as I told you, we never lend money, chiefly because such buildings are normally overbuilt.

"The term, 'taxpayer', is a misnomer and obviously a one-story building in a good store location approaches nearer in its earnings to pure ground rent than any other type of improvement."

These statements indicate how false are the economic standards on which much crowded development takes place. What justification can there be for crowding high buildings together, with all the evils of congestion incidental to this crowding, if they do not pay? As they become more crowded, they will become less profitable, owing to diminished light, air, and over-supply of space in central areas, and to the increased taxes necessary to meet expenses incidental to congestion.

Individual high buildings have their place and purpose and as long as they are made to conform to proper regulations regarding the amount of space surrounding them, they may be desirable elements in the growth of great cities. What is wrong with skyscrapers are the conditions which do not give them a proper chance to perform their functions, owing to lack of space surrounding them. The greatest enemies of concentration are those who confuse it with congestion; and the greatest enemies of high buildings

are those who deny that all buildings require adequate space surrounding them for movement and other social needs, and that the amount of space should be increased in proportion as bulk and height of building increase.

At present, in New York the assessments and taxes are higher than at any other time in the history of the city. The real estate assessment amounted to approximately $13 000 000 000 for 1926.

More than 95% of the tax permitted by law is levied. The budget for 1926 amounted to $437 000 000. The tax rate has increased in 20 years from an average 1.50 to an estimated 2.79 for 1926. The percentages of the total budget proposed to be spent include 30% for debt services and tax deficiencies, 19.9% for education, 14.7% for protection of life and property, and 1.5% for recreation, science, and art, which includes parks. The greatest increases in expenditures are for subways, policing, street cleaning, and fire prevention—all incidental to congested growth. As has been stated:* "More important than immediate economies will it be to make far-visioning plans for the future."

Financial plans will not be enough. The physical planning of the city must be made to follow lines that will lessen waste and reduce public contributions to private services. It is significant that the Planning Commission appointed by the Mayor of New York has as one of its duties that of discovering new sources of revenue.

Proposals to relieve this congested growth by construction of more subways, wider streets, bridges, tunnels, etc., put forward by responsible authorities would involve an estimated expenditure of about $2 000 000 000.

Congestion of streets is reaching the saturation point in some places. Forms of relief, such as traffic regulations, parking prohibition, narrowing of sidewalks, and street widening, are all more or less palliative. The point must soon be reached when nothing further can be done in these directions.

These and many other problems of the great city confront the regional and city planner. There are economic forces at work and scientific developments taking place that will aid in the task of influencing city development in the right directions. What then may be forecast as a reasonable prospect for the urban region of the future?

The Future Urban Region

On the whole the writer agrees with J. B. S. Haldane that science and human tendency are becoming less materialistic. It may also be assumed that the center of industrial gravity will shift to some extent as a result of the development of new sources of energy. Transportation has been the chief motive force in concentration in the past century, but it will be linked with electric energy in promoting new and more evenly distributed areas of concentration in the future. Great urban aggregations will continue to grow; and the hope lies not in restricting the quantity, but in improving the quality, of development.

* *New York World*, September 11, 1926.

The writer can visualize two forms of great city regions that will exist on this continent when the youths of to-day have entered the final laps of their race with life. Great centers like New York and Philadelphia will still be striving to maintain their supremacy, but a new kind of urban region will have sprung up to influence them and compete with them.

Just as the manufacturer has discovered the value of discarding old machinery in favor of new, the city will learn that old dilapidated buildings are a liability and that the making of great physical improvements is a sound investment. In the future old cities will be reshaping and rebuilding their blighted areas as a matter of self-preservation.

Developers of real estate will not be permitted to speculate in land, as a necessity of life. Those who sub-divide land for building will be required to make it fit for building by constructing roadways, sewers, and other local improvements in advance of the sale of the land. Playground space will have to be provided as an essential feature in any new sub-division just as street areas are at present. Building codes will require more durable forms of construction, and the cost of fire prevention will be greatly decreased. Great transportation and park corridors will radiate out from and surround the cities, giving rapid means of communication through open areas devoted to park and spacious residential areas. At strategic points along these corridors satellite cities will have grown up, connected by railroad and highway with the metropolitan center and amply provided with space for expansion of industry and healthy living conditions.

Several classes of highway will exist. There will be speedways from 120 to 150 ft. in width paralleling railroads and serving the lighter industry, business, and apartment-house areas. Interchange of freight and passengers between railroad and highway will be a common practice. The use of streets for loading and unloading of merchandise will be gradually disappearing. Owners of property will be required to carry on their business on their own premises, and streets will be more and more confined to public purposes. Buses and other highway vehicles will have properly equipped terminals as the best railroads have to-day. Parkways from 150 to 500 ft. in width, with driveways through them for rapid automobile traffic, will penetrate through areas largely used for residential purposes. In central areas elevated railroads will have largely disappeared. Air rights over many railroads will be used for elevated highways. The tendency in the centers will be to place railroads with their electric equipment underground, and to have raised sidewalks for pedestrians fronting on or arcaded into buildings. This will not add to congestion, but merely remove some of the effects of congestion created at the beginning of the Twentieth Century.

Zoning will have become more constructive and more stable. It will be based on adequate and sound standards of health and safety. Physical planning and zoning will be linked together so that, as the requirements of the law in regard to height and area of occupancy are strengthened, less necessity will arise for restricting the character of use. Factories and apartment houses will have more spacious surroundings and will fit in agreeably with adjacent

residences. The owner of the factory will have learned the lesson of the Fifth Avenue merchant that ugly ancillary features and disorderly surroundings are an injury to his business.

Interspersed between the closely developed areas along railroads and highways there will be areas zoned and permanently reserved for agricultural purposes. Regional co-operation will have ended the pollution of drinking waters and developed comprehensive systems of sewage disposal. The social needs of the community will be largely met without great loss of time in traveling, because it will be realized that there is enough land in the country to provide all facilities necessary for healthy living conditions within easy reach of every inhabitant.

The "hot dog" stand and the gasoline station will be things of the past. Just as the old English inn was a picturesque feature along the highway, so there will grow up a new kind of hostel, with pleasant surroundings where all the needs of the highway traveler can be provided.

Special ways will be set apart for pedestrians, so that walking will have become a restored art. Speedways will have bridges and subways provided for pedestrians, and separation of grades for important intersecting roads.

Motor cars will pay a proper share of the cost of highways and thereby keep up the balance between increase of vehicles and provision of highway facilities for motor travel. Rapid transit will not only pay for itself, but will yield a profit to the city for purchase of parks and playgrounds. More facilities for recreation, the results of education, and more stringent application of justice will have reduced crime.

High buildings will still be erected, but under proper regulations as to area of occupancy, so that they will have become less crowded and more in the nature of isolated monumental features of the city.

There will be a reaction against the crowding of high office buildings together and also against the apartment dwelling as a residence. A compromise will have been reached between the apartment building and the cottage which will give to a new type of home the advantages of each of the others.

Provision will have been made for numerous landing fields for aeroplanes —used for long-distance transportation and mail service. Commuting, not having increased on the whole, will be more than amply served by transit lines and improved highways. Trolleys will still operate on special rights of way, but bus services will have taken the place of many trolley lines now in use and will have become the most popular form of transit for short distances.

The more even and more open form of development, and the prevention of land speculation by irresponsible developers, will produce more stable land values. In the aggregate, land values will be no lower, but they will be more evenly distributed. Assessment of land and taxation will be more carefully adjusted to the best productive uses of land. For instance, taxes for urban improvements based on potential building values will not be levied on land permanently zoned for cultivation, or for recreation areas, such as golf courses, thereby making possible the reservation of open areas on private land.

Forty years hence New York may have its 16 000 000 or even 20 000 000 of people with less overcrowding than its present 9 000 000. Manhattan will still be the center of distribution, of exchange, of banking, and of certain industries that can be economically carried on where land is relatively expensive in the hub of the region. There will be a few more bridges and tunnels between Manhattan, Long Island, and New Jersey, but they will be planned as part of a comprehensive system that will have spread industry and population over wide areas and lessened the areas of congestion, the areas of deterioration, and the areas of waste vacant land. Many high buildings, as they have deteriorated and become out of date, will have been replaced with lower buildings, as being more profitable. Belt-line railroads, boulevards, and speedways will link together the radial system of communications. Large open areas in private ownership will be used for cultivation and recreation in the suburbs. The blighted areas along the water-front of Manhattan, Brooklyn, and The Bronx will be replaced with modern buildings giving efficient service to the port and fronting on wide streets and squares, fittingly adorned to make a dignified entrance to the city and region. The East River water-front and the Harlem River Valley will have been developed on two levels, the lower for commercial uses and the upper for monumental business and residence structures. The Hackensack Meadows will have been reclaimed and divided in use between industry, residence, and recreation. Large areas in New York City, now held out of economic use for ambitious projects that can never be realized, will have been developed, with consequent relief to formerly congested areas.

Great new railroad terminals in The Bronx, Queens, and New Jersey will have been developed, and new business centers will have grown up in their neighborhood. Long Island, The Bronx, and New Jersey will be so linked together that great volumes of traffic now passing through Manhattan will be by-passed around it. A wide parkway will extend from New York to Philadelphia. It will be intersected by a speedway for passenger cars and be paralleled by a railroad, a rapid transit line, and a speedway for mixed motor vehicles. Numerous satellite communities will be spread along this park and transportation corridor.

Great central power plants will have been developed, and power will be distributed over wide areas, serving widely scattered industrial areas.

New Regional Communities

The writer stated that the older cities will not be the only centers of the urban regions of the future. Great new centers will arise and be developed with the aid of knowledge of the past and in keeping with new opportunities. These new communities will be so developed as to cause the older communities to mend their ways.

The transmission system of the Shawinigan Water and Power Company of Quebec, Que., Canada, may be cited as an illustration. This Company, the fourth largest in the world, distributes 580 000 h.p. over an area of 15 000 sq. miles. It had a total output of 2 000 000 000 kw-hr. in 1925. The area

served is intersected by an international waterway. Within it are three large and numerous small cities—connected by an elaborate system of railroads and highways.

The defect of this area is that the great waterway of the St. Lawrence is closed for navigation for several months of the year, but it illustrates the type of area that will provide scope and opportunity for great regional communities of the future. These areas will be so large and the places suitable for efficient concentration within them will be so numerous that there will be no necessity for congestion. Large sections of land suitable for cultivation will lie between the transportation and power corridors and focal points and maintain an efficient and economical balance of development. The older city regions will gain much of their impetus to improvement from competition with these new communities.

Thus, the growth of science, the pressure of economic forces, and perhaps the growth of average intelligence, will bring into being new communities and re-shape the old in accordance with sane, practical ideals of what a great regional community should be. All that city planners can do is to help in pointing the way toward the realization of these ideals and develop their science and art so that they will be ready to make their contribution to the demands of the future.

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THE HISTORY AND DEVELOPMENT OF ROAD BUILDING IN THE UNITED STATES*

By Thomas H. MacDonald,† Esq.

Between this Sesqui-Centennial Year and the founding of the first Colonial settlement at Jamestown, Va., the great event of 1776 stands almost at the mid-point of time. The men who gathered in Philadelphia to affix their signatures to the Declaration of Independence looked back upon a period of American history a little longer than that which has since unfolded. At the end of their sesqui-centennium the journeys they made from their several homes were only slightly less arduous and time-consuming than they would have been at its beginning. To-day—150 years later—the same journeys are made in hours instead of days, and days instead of weeks.

The signers insisted that they acted for thirteen independent States. To-day, it is difficult to conceive of the States as separate Commonwealths possessing original authority and yielding only to a restricted power which they, themselves, have conferred on the Federal Government. In the popular mind they have become no more than the political subdivisions of the Republic.

To what extent was the form of government, as originally established, the product of inadequate facilities of communication between the Colonies; and what relation is there between the gradual transfer of authority to the Federal Government and the development of the railroads, the telegraph and telephone, the radio, the motor vehicle, and the modern improved highways? The writer is convinced that it would be wiser if, in the teaching of American history in the schools, more time were given to the study of such fundamental economic questions, and less to the lives and ambitions of political leaders and the wars in which America has been involved by their blundering.

It is an absorbing pursuit in reading history to speculate on the effect of transportation in shaping America's destiny as a nation. To what extent

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was the general improvement of roads that followed immediately after the Revolution owing to the preservation and gradual strengthening of the bond which precariously united the Thirteen Colonies?

What seeds of disunion are to be found in the fact that the later movements of the people and the building of roads, canals, and railroads were mainly westward both from the South and the North; and that between the Hudson and the Mississippi not a single major waterway runs north and south?

What would now be the western boundary of the United States if exploration and travel over the Oregon Trail had been delayed by as much as a decade?

What conception of the true rôle of the railways was held by those who built the early roads? The writer is convinced that they thought of them as merely a new form of highway, to replace entirely the common roads. No other conception and no less profound ignorance of the economics of railway transportation can explain the hundreds of thousands of short lines built without connection or possibility of adequate revenue; and no other conception will account for the complete abandonment of the common roads.

What importance as an underlying cause of the Civil War attaches to the fact that, in 1850, there were less than 10 000 miles of such inadequately located railways in the entire country and, in 1860, but slightly more than 30 000 miles? And what, in view of the general agreement that the South was starved into submission, was the effect of the fact that, in 1864, there were less than 6 000 miles of railway under Confederate control, all in utter disrepair?

These are interesting historical speculations. In the forces with which they are concerned is found a truer cause of the course and progress of the Nation than many of those of a purely political nature to which such importance has been attached by historians.

Amusement as well as intellectual profit may be derived from this study of the history of transportation. Whenever opposition is expressed to the granting of franchises for the use of the public highways by common carrier motor trucks, the writer is reminded of the completeness with which the Governor of New Jersey disposed of a similar complaint by the Assembly in 1707. Of the result of this so-called monopoly he stated:

"At present everybody is sure, once a fortnight, to have an opportunity of sending any quantity of goods, great or small, at reasonable rates, without being in danger of imposition; and the sending of this wagon is so far from being a grievance or monopoly, that by this means and no other, a trade has been carried on between Philadelphia, Burlington, Amboy, and New York, which was never known before, and in all probability never would have been."

Let the legal retainers of the public service commissions strive as they may, there is no more adequate reply to the numerous complaints, which it is their duty to answer, than that.

When one hears the fear expressed, as frequently happens, that the improvement of roads and their use by motor vehicles will destroy the business of the railroads, the writer is again reminded of the instance cited by a

citizen of Pennsylvania in an essay he prepared in 1891 in the hope of winning a prize offered by the University of Pennsylvania. He was informed, so he states:

"By a gentleman in the business of furnishing road material, that a prominent official of a great railroad refused to deliver material for him at a point on their road where it would have been a great convenience to have had it, and gave as a reason that they did not wish to accommodate the turnpike company that wanted the material, as their road was too good already, and was competing with the railroad to the injury of their business."

If, in 1891, before ever the honk of a motor horn had disturbed the quiet somnolence of the peaceful Pennsylvania countryside, a "prominent official of a great railroad" could scent danger in highway improvement, it is not surprising in this day of greatest railroad prosperity, that other "prominent officials" betray similar fears, but it is problematical whether they still hope to halt the proper development of highway transportation.

These are merely random historical allusions by way of preface to the specific subject of this paper, which is confined to a review of the development of highway construction and maintenance.

Colonial Road Building

With the exception of the primitive streets of Jamestown no roads were built in the Colony of Virginia by the first settlers. The settlements were close to the river banks and fear of Indian attack kept the colonists from wandering far inland. The first road law of the Colony, which was also the first American law with respect to highways, was passed by the General Assembly in 1632. It provided for the laying out of the simple ways required by the small settlement in the following terms:

"Highways shall be layd out in such convenient places as are requisite accordings as the Governor and Counsell or the Commissioners for the monthlie corts shall appoynt, or accordings as the parishioners of every parish shall agree."

Brief and elementary as were its terms, this law was doubtless entirely adequate for a community in which, according to the inventory of goods and chattels made in 1625, there was but one horse, and that in the possession of the Governor.

The first New England law of record, and the second to be passed in all America, was enacted in Massachusetts in 1639. Although somewhat longer than the Virginia Act, its terms were similar in their general effect. It is notable from the point of view of the engineer, however, as the first law in which there is any specification as to the manner in which the roads were to be constructed. After providing for the laying out of the roads by two or three men chosen from every town, and after carefully protecting the rights of property owners from encroachment by the road builders, it lays upon the locators this single injunction, that, "in common grounds, or where the soil is wet or miry, they shall lay out the ways the wider, as six, or eight, or ten rods, or more in common grounds."

In this law is found the first provision, loose and flexible as it was, governing the width of the highways. As yet there was no definition of the manner in which the ways were to be made; and it was not until 1664 that, in the regulations established by the Government of New York, the first definite prescription of this nature was given. That part of the regulation was, as follows:

"The highways to be cleared as followeth, *viz.*, the way to be made clear of standing and lying trees, at least ten feet broad; all stumps and shrubs to be cut close by the ground. The trees marked yearly on both sides—sufficient bridges to be made and kept over all marshy, swampy, and difficult dirty places, and whatever else shall be thought more necessary about the highways aforesaid."

Although this was the first legal definition of a method to be followed, it is probable that the regulation merely embodied the common practice of the day and that methods and practices very similar were followed in all the Colonies.

That but little advance had been made by the opening of the Eighteenth Century is indicated by the Maryland Law of 1704, which contains much the same provisions with respect to clearing and grubbing, and merely increases the width to 20 ft. This law, however, is of interest mainly because of its Fifth Section which constitutes what is perhaps the first definite regulation for marking of roads in America, and, incidentally, indicates how easy it must have been to lose one's way on the rude tracks which served for roads. This curious provision was as follows:

"V. And that all the roads that lead to any Ferries, Court-house of any County, or to any church, or leading through any county to the port of Annapolis, should be marked on both sides the Road with Two Notches; if the Road lead to Annapolis, the Road that leads there, at the leaving the other Road, shall be marked on the face of the tree, in a smooth Place cut for that purpose, with the letters A A set on with a Pair of Marking-Irons and coloured; and so with Two Notches all along the Road; and where at any Place it leaves any other Road shall be again distinguished with the Mark aforesaid, on the Face of the Tree, with a Pair of Marking-Irons, and coloured as aforesaid. And any road on the Eastern Shore in Talbot County, that leads to the port of William-Stadt at the entering of the same, and in parting with or dividing from any other Road, shall be marked on the face of a tree, in a smooth place cut for that purpose, with the letter, W, and so with two notches all along the road. And the Roads that lead to any County Court-house, shall have two Notches on the Trees on both sides of the Road as aforesaid, and another Notch a Distance above the other. And any Road that leads to a Church, shall be marked at the entrance into the same, and at the leaving any other Road, with a Slip cut down the Face of the Tree, near the ground. And any road leading to a Ferry, and dividing from other public roads, shall be marked with Three Notches of equal distance at the Entrance into the same. And these Rules and Methods the Several Justices of the County Courts, shall, from Time to Time, give in charge to the overseers of the Highways, by them to be appointed for that purpose; who are likewise enjoined carefully and strictly to observe and perform the same, under the Penalty aforesaid."

Along at least one of these ancient roads in Southern Maryland, still known as the "Three Notch Road", notches, some of a remote date, may still be discerned in the bark of many of the oldest oaks.

The law in which this odd provision was made constituted, with minor amendments, the sole road legislation of the Province of Maryland for nearly three generations, and Maryland was not more backward than the other Colonies. Indeed it must be said that from the point of view of the road builder the progress made during the entire Colonial period was practically negligible, as may be seen from the following passages taken from the work of the Maryland historian, Thomas Scharf:

"* * * where water routes were not available, the means of locomotion, though various, were all primitive. Long journeys were made on horseback. People of consequence rode in their coaches, with four horses attached, the leaders mounted by liveried postilions. So Washington came from New York to Annapolis, to attend the ball given there in his honor after the peace. He left New York, December 4, and arrived at Annapolis, December 17, thirteen days to a ride which may now be made inside of eight hours. So Washington went to Philadelphia from Mount Vernon to attend the Constitutional Convention, having to lie over at Havre-de-Grace all night because it was too stormy to cross the Susquehanna. The stage-coach was just coming into use in this country at the time of which we write, and the importance of regular communication between one point and another was beginning to be seen. There were a few, but not many post-routes, and those chiefly maintained by private enterprise.

"The carrier's cart plied between Alexandria and Philadelphia, by way of Baltimore; the Conestoga wagon was the means of communication between Baltimore and Harrisburg, Frederick, Hagerstown, etc., while these outlying places in their turn were brought into intercourse with the backwoods and the wilderness by means of strings of pack-horses. * * * Roads were all bad and ill-kept, narrow, obstructed by gates and seldom permitting two vehicles to pass one another. * * *

"The good roads—comparatively good, we mean—and the best post-routes were due to private enterprise. The Kent Island post-route was owned by a Tilghman, and Henry Callister threatened an opposition unless it was better managed. * * * It took a day of almost twenty-four hours, to ride from Elkridge Landing to Annapolis (about 30 miles)—yet the thrifty Germans of Frederick County traded with Georgia by way of the Valley of Virginia, sending their manufactures of wool, flax and leather on the backs of pack-horses."

Pioneer Roads to the West

If such was the state of the roads in the settlements during the Revolution it may be assumed that the few roads which previously had been pushed westward across the mountains were even more primitive. The first of these was laid out in 1743 by the Ohio Company, a party of Virginia gentlemen who had been granted 200 000 acres on the Ohio River between the Monongahela and the Kanawha on condition that 100 families should be settled on it in two years.

According to Hulbert:

"The road was probably nothing more than a blazed trail with possibly some alteration of route at certain points; it may be that in low ground which could not be avoided the road was 'corduroyed', but this is quite doubtful."

It was over this road that Washington traveled, in 1753, at the behest of the Virginia Governor to inquire of the French their rights to build forts on the Allegheny River. Again, in 1754, when war with the French seemed inevitable, he made the same trip over the same road, this time taking with him 300 men and possibly 10 swivel guns, to move which it was necessary to widen and straighten the road. Finally, in the following year (1755), it was over this same road that Braddock advanced from Fort Cumberland with 2 200 men and 600 pioneers, leading the way, to widen and corduroy the trail for the Army. This time the road was evidently made with some care, for Washington complained that the movement of the army was being delayed because the road-makers were stopping to level "every mole-hill".

This, the first road across the mountains, was the only one until 1758, when Gen. John Forbes, sent against Fort Duquesne to redeem the failure of Braddock, built or rather opened a new one by way of Bedford and Ligonier, in Pennsylvania; and this road with its predecessor remained as the only routes of white-man travel across the mountains until 1775.

In that year the Transylvania Company was organized, and Daniel Boone was sent to mark out the Wilderness Road through the Cumberland Gap to Kentucky. Although made later than the roads of Braddock and Forbes, the Wilderness Road quickly became the most important of the three as a thoroughfare and for a time actually outrivaled the Ohio River as a pioneer route. Of this road a Kentucky historian says:

"The road marked out was at best but a trace. No vehicle of any sort passed over it before it was made a wagon road by action of the State legislature in 1795. The location of the road, however, is a monument to the skill of Boone as a practical engineer and surveyor. It required a mind of far more than ordinary caliber to locate through more than two hundred miles of mountain wilderness a way of travel which, for a hundred years, has remained practically unchanged, and upon which the State has stamped its approval by the expenditure of vast sums of money."

Although the State did undoubtedly approve and profit greatly from the work of the intrepid Boone, its approval apparently did not express itself in any substantial way, for in 1796, he wrote as follows to Governor Shelby:

"Sir, after my best Respts to your Excellency and famyly, I wish to inform you that I have sum intention of undertaking this New Rode that is to be cut through the Wilderness, and I think my Self intitled to the ofer of the Bisness as I first Marked out that Rode in March 1775 and Never rec'd anything for my trubel and Sepose I am no Statesman I am a Woodsman and think my Self as Capable of Marking and Cutting that Rode as any other man. Sir if you think with Me I would thank you to wright me a line by the post the first oportuneaty and he Will Lodge it at Mr. John Milers on hinkston fork as I wish to know Where and when it is to be Laat [let] so that I may atend at the time I am Deer Sir your very omble sarvent.

"DANIEL BOONE."

But, alas, the contract was given to others, to Boone's great disappointment. Such was the experience of the first American "engineer" who took it into his head to "go into contracting." How many have been the similar experiences of his successors the record does not state.

The mountains crossed, progress of exploration westward was rapid after the Revolution. The pioneers opened the trails and the Army followed close behind, establishing forts for the protection of travelers and settlers against Indian attack. The "road building" that was an incident of the advance of those brave and adventurous spirits, however, was invariably of the sort already described; that is, a mere breaking and blazing of trails in the difficult places, and, in the open country, only the choosing of a path which by continued use was worn into a well-marked route.

For the opening of certain of these routes, Congress, which under the Constitution was given the right to govern the territories, was urged to contribute money, and did actually contribute to a few, among them the famous Natchez Trace which ran from Nashville, Tenn., to Natchez, Miss., and Zane's Trace in Ohio. The character of the work done on the former is indicated by the fact that the appropriation was $6 000.

Of similar character were the great trails of the West, on which exploration began early in the Nineteenth Century. In all the annals of America there are no more thrilling passages than the records of the travels and the path-finding of the pioneers who, following the Santa Fé, the Overland, and the Oregon Trails, settled and held for the United States the great dominions of the West. The highways they marked out were so truly remarkable that they are still, in the main, the principal routes of travel in the territories they traverse. One of them—the Oregon Trail—has been described as:

> "The most remarkable road known to history * * *. Considering the fact that it originated with the spontaneous use of travelers; that no transit ever located a foot of it; that no level established its grades; that no engineer sought out the fords or built any bridges or surveyed the mountain passes, that there was no grading to speak of, nor any attempt at metalling the road bed; and the general good quality of these two thousand miles of highway will seem most extraordinary."

Early Turnpike Ventures

It will not be possible, however, in this paper to follow the development of the routes of exploration westward. The principal object being the study of the development of road construction methods, the writer must return to the Eastern States, where, immediately after the close of the Revolution, the first great movement for the betterment of American roads was at its inception—the establishment of the turnpikes.

The first American turnpikes of which there is record were established in Virginia, Connecticut, and Maryland. From the interesting book by Frederic J. Wood entitled "The Turnpikes of New England", it is learned that Virginia led the way by the erection of gates on the roads leading into Alexandria from Snigger's and Vesta's Gaps. The travel on these roads had become so heavy that extensive repairs had become imperative and, the resources of the territory traversed being inadequate, it was proposed to set up toll-gates as a means of collecting the necessary revenues, an idea suggested, no doubt, by the English experience with turnpike trusts which had been in operation for at least twenty years previously. The Act of the

Legislature providing for the erection of the turnpikes was passed in 1785, and, as an existing road was taken over and the only construction necessary was the erection of the gates, it is safe to say that they were in operation by 1786.

A heavy travel passed over this road for several years, and the tolls imposed by the Legislature were insufficient to maintain its surface. Accordingly, in 1795, representations were made to the Legislature that "an artificial bed of pounded or broken stone" was necessary, and that the expense of this, being too great to be financed by the usual methods, it should only be assumed by private enterprise. This is the first reference the writer has been able to find to any form of construction resembling what is now classed, very loosely, as a macadam surface. It is probable that stone and gravel surfaces had been laid in England for some years previously, and there may have been some more or less unsystematic use of stone in America also at an earlier date; but it is reasonably certain that this Virginia road was at least one of the earliest to be deliberately planned as a stone-surfaced highway. As a result of the representations made, the "Fairfax and Loudon turnpike-road company" was incorporated on December 26, 1795, and given the privilege of reconstructing the old turnpike from "Little River, where the present turnpike crosses it, to Alexandria." The first company finding the task too great, nothing was accomplished under its charter, and, in 1802, a new charter was granted to the "President, Directors, and Company of the Little River Turnpike Company." Under this charter, with the help of the State of Virginia, which appropriated the "muster fines" in 1805 to the purchase of 100 shares of stock, the old road was rebuilt and operated as a toll road for more than ninety years.

The second and third turnpikes were in Connecticut where, in 1792, the Mohegan Road, between New London and Norwich, and the Old Post Road, in Greenwich, were made subject to toll. A year later, in 1793, a gate was set up on the Reisterstown Road, in Maryland and, later, other gates were established on the York and Frederick Roads near Baltimore. These Maryland cases were followed, in 1801, by the establishment of a gate on the old road through Cumberland Gap, in Tennessee; and, in 1804, North Carolina provided for the construction of a 14-mile road through the Cherokee lands by granting to the builder the privilege of collecting tolls for fifteen years. All these were Government enterprises, and they are the only exceptions to the practice, which was shortly to become general, of allowing the turnpikes to be built by private capital. Later, these, too, were transferred to private companies, but the first road to be built in this manner was the famous Lancaster Turnpike, in Pennsylvania, which was built by the Philadelphia and Lancaster Turnpike Company, incorporated by the Governor on April 9, 1792. This road which was more than 62 miles in length, was built at a cost of $7 450 per mile and was finally completed in 1796.

Following this example similar corporations were created in rapid succession in all the States, and in his report to Congress, in 1808, Albert Gallatin,

the Secretary of the Treasury, was able to report definitely that 770 miles were completed in Connecticut, more than 3 000 miles were under construction or completed in New York, and hundreds of miles in other States.

The Character of Early Turnpikes

This same report of Gallatin's gives the first detailed description of the character of the "artificial roads" built by the early turnpike companies.

Referring to the road from Trenton to Brunswick, in New Jersey, he reports that:

"The distance is twenty-five miles; the greatest angle of ascent, three degrees; and the road is nearly in a straight line, the only considerable obstruction being the 'sand hills', through which it was necessary to dig at the depth of thirty feet, in order not to exceed the angle of ascent. The road is thirty-six feet wide, fifteen feet of which are covered with about six inches of gravel. A few wooden bridges, with stone abutments, and piers have been erected across the intervening streams. The whole expense is stated at $2 500 a mile."

On the road in Pennsylvania from Philadelphia to Perkiomen, which had two branches to Willow Grove and Chestnut Hill, he wrote:

"The distance * * * is twenty-five miles and a quarter; the two branches extend one ten miles, and the other seven miles and a half; making together near forty-three miles. The angle of ascent is four degrees; the breadth of the road fifty feet, of which twenty-eight feet, having a convexity of fifteen inches, are covered with a stratum either of gravel eighteen inches thick, or of pounded stones twelve inches thick. One-half of the stones forming the lower part of the stratum are broken into pieces not more than five inches in diameter; the other half or upper stratum consists of stones broken into pieces not more than two inches and a half in diameter, and this difference in the size of the stones is represented as a considerable defect. Side or summer roads extend on each side of the gravel or stone road. The five miles next to Philadelphia have cost at the rate of $14 517 a mile; the other twenty miles and a half at the rate of $10 490 a mile. Yet there were no natural impediments, and only small bridges or culverts were necessary. The capital expended on these twenty-five miles and a half is $285 000; the tolls amount to $19 000; the annual repairs and expenses to $10 000; the nett income to about $9 000, or a little more than three per cent. on the capital expended."

Among Maryland roads he notes the Reisterstown Turnpike, 24 ft. wide and surfaced with 12 in. of pounded stone not more than 3 in. in diameter; the Frederick Road, 22 ft. wide and surfaced with 10 in. of pounded stone not more than 3 in. in diameter, over which were spread 2 in. of gravel or coarse sand, the entire surface having a crown or convexity of 9 in.

South of the Potomac Gallatin reports that few "artificial roads" had been undertaken. One, which he briefly describes—the road from Manchester to the coal mines of Falling Creek, in Virginia—was 36 ft. wide and graveled and the cost of 12 miles he gave as $50 000. This construction, he reports, was sufficiently substantial to admit wagons carrying 4 tons.

"The greater progress made in the improvement of roads in the northern parts of the Union," he ascribes, "to a more compact population, which renders those improvements more necessary, and at the same time supplies with greater facility the means of effecting them. The same difference is perceptible in the number of bridges erected in the several States."

In this famous report is direct evidence of the construction, prior to 1808, of roads surfaced with both gravel and broken stone. The Philadelphia-Perkiomen Road was constructed of a stone surface in two courses in a manner similar, except for the large size of the stone and the heavy crown and the absence of rolling and sprinkling, to the methods used in building modern macadam roads. Wherein lay the "defect" of using two courses, as mentioned in the report, is not made clear.

MacAdam's Principles

It is not to be assumed, however, that these roads embodied in their construction the principles of MacAdam; nor can it truthfully be stated that present roads do. They were probably patterned after the roads then being constructed in England, and the great road builder, whose name is now associated with any form of stone surface, had not yet demonstrated the particular methods which properly should be attributed to him .He was still engaged in the preparatory studies which led to the development of the methods he first applied when he became the Surveyor of Roads in Bristol in 1816.

According to MacAdam's view, the function of the stone surface was not to support the weight of vehicles, but to form a covering or roof over the natural soil on which, in its dry condition, he relied entirely to support the load.

He, therefore, regarded a heavy course of metal as unnecessary, and large stone as positively objectionable in that a road so formed would be likely to be more pervious than one made of small stone. His idea was that, "every piece of stone put into a road, which exceeds an inch in any of its dimensions, is mischievous"; and he based this conclusion not only on the thought that the use of larger stone would increase permeability, but, also, and perhaps mainly, on the greater tendency for the larger stones to be dislodged by the wheels of vehicles. He was also opposed to the use of heavy crowns on the ground that they were unnecessary for drainage purposes and that they tended to concentrate traffic at the center and, thereby, actually defeat the ends of drainage by causing the formation of ruts.

His methods, explained in his own words, were as follows:

"As no artificial road can ever be made so good, and so useful as the natural soil in a *dry state*, it is only necessary to procure and preserve this dry state of so much ground as is intended to be occupied by a road.

"The first operation in making a road should be the reverse of digging a trench. The road should not be sunk below, but rather raised above, the ordinary level of the adjacent ground; care should at any rate be taken that there be a sufficient fall to take off the water, so that it should always be some inches below the level of the ground upon which the road is intended to be placed: this must be done, either by making the drains to lower ground, or if that be not practicable, from the nature of the country, then the soil upon which the road is proposed to be laid, must be raised by addition, so as to be some inches above the level of the water.

"Having secured the soil from *under* water, the road-maker is next to secure it from rain water, by a solid road, made of clean, dry stone, or flint, so selected, prepared, and laid, as to be perfectly impervious to water; and

this cannot be effected, unless the greatest care be taken, that no earth, clay, chalk, or other matter, that will hold or conduct water, be mixed with the broken stone; which must be so prepared and laid, as to unite by its own angles into a firm, compact, impenetrable body.

"The thickness of such a road is immaterial, as to its strength for carrying weight; this object is already obtained by providing a dry surface, over which the road is to be placed as a covering, or roof, to preserve it in that state: experience having shewn, that if water passes through a road, and fills the native soil, the road, whatever may be its thickness, loses its support, and goes to pieces."

It must be borne in mind that MacAdam applied his art more often to the repair of existing surfaces than to the construction of new ones, and in making such repairs he particularly recommended and used the process which he called "lifting the road" which consisted in loosening the upper portion of the surface and breaking all old stones of more than about 6 oz. in weight before applying the new stone. As he did not have a roller he advocated applying the new stones in thin courses, allowing time between successive applications for the metal to be compacted by traffic. He suggested a three-time application as desirable and particularly insisted on the necessity of keeping the road well shaped during the intervals when it was being compacted.

He surprised the members of the Parliamentary Committee appointed to inquire into methods of improving the turnpikes and highways by informing them that he would prefer to construct his surface on a soft foundation rather than a hard one, because "when a road is placed upon a hard substance, such as a rock, the road wears much sooner than when placed on a soft substance".

This statement elicited from his amazed interlocutor the exclamation: "You don't mean you would prefer a bog?" to which the calm reply was; "If it was not such a bog as would not allow a man to walk over, I should prefer it."

With these views, so remarkably in accord with the most modern ideas, there is scarcely a fault to be found, yet the fact remains that they have been honored more in the breach than in the observance in American practice, at least.

As MacAdam did not begin to practice his art until 1816 and as his methods were essentially different from those common at the time in England, it is not possible that the stone surfaces constructed on the fast multiplying American turnpike roads prior to that date were macadam roads in the strict application of the term, although, from the descriptions available, it is probable that they were not unlike many of the roads to which the name is now applied.

When, in 1832, it became necessary to repair the surface which had been constructed on the Cumberland Road east of the Ohio River, the orders of General Gratiot, then Chief of Engineers, were that the macadam system was to be followed, and this system he defined exactly as MacAdam had described it in his "Remarks on the Present System of Road Making", published in 1820. It is not improbable that this was the first application of the true method in America, and it may have been the last.

The National Pike

The story of the National Pike has been told too many times to need more than a brief reference herein. The earliest Act of Congress relating to it was passed in 1802 and simply provided that 5% of the revenue received from the sale of Ohio lands should be set aside for public road improvement and that two-fifths of this amount should be devoted to constructing a road from the Potomac River westward to the Ohio River.

In 1805, the building of the road from Cumberland, Md., to the Ohio River was authorized, and the Act further provided that the proposed road was to have a right of way of 4 rods, a roadway of 20 ft., a maximum grade of 8¾%, and a surface of stone, earth, gravel, or sand, or a combination of these. Later, in Ohio, the width of right of way was increased to 80 ft.

The first contract for construction was let on April 11, 1811, and the road was completed from Cumberland to Wheeling, W. Va., about ten years later. The total length of this section is 131 miles and the total cost of construction was $1 706 845, or an average of $13 029 per mile. The entire section east of Wheeling was surfaced with broken stone put down in two courses. The specifications provided that the stone for the bottom course should pass a 7-in. ring, and that for the top course, a 3-in. ring. The top course was 6 in., and the bottom, 12 in., before compacting, and as the traffic was depended upon to effect the compacting, there is little doubt that travel over the road in springless wagons was not unduly comfortable, especially as it is said that the engineers in charge of the work had a way of erecting barriers which were frequently shifted in order to prevent honest travelers from using any part of the surface after it had become compact.

The first appropriation for continuing the "National Road" west of the Ohio River was made in 1825, and the final appropriation for completing it in Ohio and continuing it in Indiana and Illinois was made in 1838. The section in Indiana was constructed east and west from Indianapolis, and was practically completed before the Government discontinued its appropriations, but in Illinois no surfacing was laid. Congress provided that only grading should be done on this section, in order that the Government might later put down a steam railroad on the grade if it so desired.

Turnpike Companies the First Public Service Corporations

The turnpike-road companies, by which the majority of the early artificial roads were built, were the first public service corporations. They operated under charters granted by the State Legislatures, authorizing them either to build and maintain an entirely new road or to take over from the public the maintenance of an existing road.

What now seem to be very severe restrictions were imposed upon them. They were limited strictly to the building and maintaining of a road, and were not allowed to do anything else. Rates of toll were fixed in the charter, and the number of gates to be erected was also specified. The location of the road was not entrusted to the judgment of those who were investing their

money, but was delegated to a committee appointed either by the Legislature or by the Judge of the County Court.

In spite of these restrictions the stock of the companies at first was very attractive to investors; but it was not long before it was discovered that the roads could not be expected to pay a large return. Few of the companies were able to show, even at first, net earnings of more than 2 or 3%, and as time went on and the cost of maintenance increased, and as, finally, the railroad came to take away a large part of the traffic, even these small profits vanished, and the turnpikes became, generally, very unprofitable investments. Most of the hundreds of companies that were organized had failed and the roads had reverted to public control by 1850; but a few remained in operation, after a fashion, until very recent years, their roads anathema to all who were compelled to use them.

The word "turnpike" has come to be associated with a stone surface, mainly because the surviving "pikes" were all, or practically all, of that character. In the early days, however, many of the companies, perhaps the majority, built only a graded road. On many roads the surface was of gravel, in others, shell, and in some, in order to make the roads passable, the companies resorted to corduroy. Quite a large number, as will be noted later, were incorporated for the special purpose of building plank roads.

The railroads sounded the knell of the turnpike companies. One by one they came on financial difficulties so great as to cause them to give up their charters, until, by 1850, few were left in existence. With their passing the roads reverted to public control—which meant county control—and so out of the picture until the late Eighties, except for the rather curious vogue of the plank road.

The Plank Road Epidemic

The first plank road on the continent, and probably in the world, was laid at Toronto, Ont., Canada, in 1835-36. The originator is believed to have been Dr. Darcy Boulton. About ten years later, a road was built, in all respects like the Toronto example, at Syracuse, N. Y. This was probably the first road of its kind in the United States. Within four years more than 2 000 miles had been constructed in New York State alone, at an average cost of $1 833 per mile. Several were built, also, in New Jersey, and a number in Vermont and Connecticut. At least one was built in Massachusetts. These, so far as the record goes, were the only Eastern developments; but the idea spread to the interior States and there the planked surfaces may still be found occasionally.

Of the construction of the plank roads, Mr. Frederic Wood*, writes as follows:

"They were in nearly all cases of single track, laid on the right side of the road as one faced the large town to which it led. In the prairie regions the planking lay on the original surface of the ground, but in some places a small amount of grading was needed to avoid short, steep ascents. The sub-

* See also "The History, Structure, and Statistics of Plank Roads," published in 1850, by W. Kingsford.

grade once established, longitudinal trenches were dug in which sills consisting of 3-inch plank four and eight inches wide were placed, and on them were laid the planks, three inches thick and eight feet long, at right angles to the direction of the road. The sills were set slightly below the surface of the ground, and the planks were pounded down to rest upon them by means of a large mallet known as a 'commander'. No nails or pins were then needed to hold the planks down, and it is reported that it was hard work to take one of them up. After the planks were laid the earth was packed against their ends and soundly tamped into place. The portion of the highway not occupied by the plank road was usually maintained as a common dirt road and was locally known as the 'turn-off', because light loads had to leave the planks and follow it when passing a team proceeding the other way. In order that a wagon might regain the planked surface without its wheels sliding along the edges, the planks were staggered, that is, one-half of them had their ends in a straight line with each other, while the other half were alternately advanced to a line six inches farther out, producing a border effect like the battlements of a castle. Ordinarily, two stringers were used but occasionally three. Over the completed planking a layer of sand was spread and maintained, which preserved the road by reducing the cutting by the calks of the horseshoes. It was claimed for this coating that a saving of forty to fifty per cent was secured in the wear of the road. Very few double-track roads were ever laid, and in the few cases it was preferred to lay two single tracks, apart from each other and supposedly on opposite sides of the 'turnout'.

"Yellow pine was preferred for the planking in Central New York, on account of its durability and freedom from knots, but hemlock and white pine were extensively used while beech, maple, and elm were employed in some places. * * * Of course the planks would rapidly decay, and it was considered necessary to figure that a road would have to be rebuilt at the end of every seven years."

The Development of Modern Types of Pavement

Passing immediately to the later Eighties no development of any importance will be missed so far as the highways are concerned, except the inventions of the stone crusher by Blake in 1858 and of the steam road roller by Lemoine in 1859. The former, at least, was apparently not received with enthusiasm, because the author of a prize-winning essay wrote in 1890 that:

"Hand-broken stone is much superior to that crushed by a machine, which is generally of irregular shape and seldom cubical, so that it does not readily bind together, which is the essential qualification of macadam."

Some time during this period, probably after 1876, the third, or binder, course was added to the macadam process. In his book entitled "A Practical Treatise on Roads, Streets, and Pavements", published in 1876, the late Q. A. Gillmore, M. Am. Soc. C. E., makes no reference to it, except as a top dressing of gravel or earth spread on after the rolling of the last application of stone. This is the method practiced even in MacAdam's day of which he strongly disapproved. The consolidation of the road with stone chips flushed and rolled into the surface was apparently unknown to Gillmore when the first edition of his book was printed; but some time between 1876 and 1890 it became the common practice.

It was during the latter part of this same period (1850-90) that the first brick and sheet asphalt surfaces were laid; the brick in Charleston, W. Va.,

in 1872, and the first extensive sheet asphalt surface on Pennsylvania Avenue, in Washington, D. C., in 1879. Prior to 1879 the first rock asphalt surface had been laid opposite the City Hall in Newark, N. J. This was in 1870; and there are conflicting claims to priority in the construction of short experimental sections of sheet asphalt, one being that the first section was laid in New York in 1873 and the other that it was in Washington Square, Newark, in 1877. For many years, however, construction of both the brick and asphalt surfaces was confined to cities. Until 1890, at least, the only materials generally considered as practical for surfaces of rural highways were broken stone, gravel, and—where the materials were available—shell and slag. The latter were laid in the same manner as gravel and stone. The plank surface was already condemned. Surfaces of "burnt clay"* had been suggested, but were still untried.

The organization of the League of American Wheelmen in 1880 may probably be considered as the turning point marking the beginning of the modern period of highway improvement, and the League itself as the first of the influences which have led to so great a change in the public attitude. One of the principal objects of the organization was to secure better roads, and it worked energetically to attain its purpose; but it was not until eleven years later that the first tangible result of its efforts was accomplished in the passage of the first State-Aid Law in New Jersey. Writing in 1889, Jeremiah Jenks† pictures the condition existing at that time when he states that:

"A very large proportion of our people, too, have never seen a really good road for hauling purposes, and have, in consequence no clear idea of the gain that would come from good roads."

The Renewal of State and Federal Participation

The passage of the New Jersey State-aid legislation in 1891 was the first practical step taken, anywhere in the United States, to remedy the situation. With one exception, it was also the first instance in which any State had undertaken to participate directly in the construction of roads. The exception is Kentucky, which had a State Highway Department and a well-defined State road policy from 1821 to 1837, and during that period completed upward of 340 miles of roads.

The next fruit of the League's efforts is found in the establishment of the United States Office of Road Inquiry in accordance with an Act of Congress approved March 3, 1893. The purpose of this Office, as defined in the statute, was to make inquiries in regard to systems of road management throughout the United States; to make investigations in regard to the best methods of road making; to prepare publications, and to assist agricultural colleges and experiment stations in disseminating information, on the subject. The appropriation made for this purpose was $10 000.

It will thus be seen that the renewed participation of the Federal Government after a lapse of fifty-five years was limited to activities of an educa-

* "Road Making and Maintenance," by Charles Punchard, published with other essays in "A Move for Better Roads," Univ. of Pennsylvania Press, 1891.

† "Road Legislation for the American State," by Jeremiah W. Jenks.

tional character, but these it must be said were carried on with excellent judgment and the utmost vigor possible with the meager appropriation provided.

The actual construction of the roads being still under the control of county and township authorities, the Federal Office carried its message directly to these local officials and by actual demonstration on short sections of object-lesson road in every State, taught them the methods of road construction which, from its studies, it found to be best. The general public it reached through the medium of bulletins, lectures, and good roads trains, and there is no doubt that its urging was responsible for the creation of more than one of the early State highway departments.

Following the New Jersey precedent, laws providing for the establishment of such departments and the granting of State aid were passed in Massachusetts in 1892, in California and Connecticut in 1895, and in Maryland, New York, and Vermont in 1898. Thus, by the end of the Nineties there were seven States—all but one on the Atlantic seaboard—that had taken the initial step toward the solution of the road problem; and as the century opened they were followed by many others. The dates of passage of the first laws in the several States are shown in Table 1.

TABLE 1.—Dates of Passage of State-Aid Highway Laws.

State.	Year in which first State-aid law was passed.	State.	Year in which first State-aid law was passed.
Alabama	1911	Nevada	1911
Arizona	1909	New Hampshire	1903
Arkansas	1913	New Jersey	1891
California	1895	New Mexico	1909
Colorado	1909	New York	1898
Connecticut	1895	North Carolina	1901
Delaware	1903	North Dakota	1909
Florida	1915	Ohio	1904
Georgia	1908	Oklahoma	1911
Idaho	1905	Oregon	1913
Illinois	1905	Pennsylvania	1903
Indiana	1917	Rhode Island	1902
Iowa	1904	South Carolina	1917
Kansas	1911	South Dakota	1911
Kentucky	1912	Tennessee	1915
Louisiana	1910	Texas	1917
Maine	1901	Utah	1909
Maryland	1898	Vermont	1898
Massachusetts	1892	Virginia	1906
Michigan	1905	Washington	1905
Minnesota	1905	West Virginia	1909
Mississippi	1915	Wisconsin	1911
Missouri	1907	Wyoming	1911
Montana	1913	....	....
Nebraska	1911	....	....

The State-aid policy as adopted by the various States took a number of forms. In some States the aid offered consisted only of advice which might be accepted or rejected by the local authorities who retained absolute control over all the roads. In such States no financial aid was extended. In those States which provided for financial aid its acceptance generally implied an agreement on the part of the county to accept the supervision of the State

authorities until the work of construction was completed, after which the road reverted to full county control. In still other States the joint participation of the State and county in the construction of certain classes of roads, generally the most important ones, was made mandatory; and there were still other variations which differentiated the systems adopted by the various States.

Road Conditions in 1904

While the pioneer States were making this radical departure from long established methods of highway administration, certain seemingly unimportant and unrelated events were occurring in various parts of the country. Three men—whose names were Duryea, Haynes, and Ford—had made themselves horseless carriages; and in Bellefontaine, Ohio, a road had been built entirely of concrete. Concrete had been used as a base for block and sheet asphalt pavements for many years; but the short section built in 1893 in the Ohio city was the first to be built with concrete as the wearing surface. As with brick and asphalt surfaces, it was several years, however, before use of the new type was extended to the rural highways; and the same may be said of the automobiles. Although their numbers increased remarkably in the cities, it was several years before they became a problem to the rural highway builder.

In 1904, the U. S. Office of Public Roads, lineal descendant of the U. S. Office of Road Inquiry, took the first census of American roads. It found that there were then, in the United States, 2 151 570 miles of rural highways, of which 153 662 miles had been surfaced with various materials. Of these surfaced roads, 38 622 miles were improved with water-bound macadam; 114 899 miles with gravel, sand-clay, shell, plank, and other low-type surfaces (the great majority being gravel); and 141 miles with surfaces better than macadam. The last included 123 miles paved with brick, of which 104 miles were in the two States of Ohio and West Virginia, and 18 miles of bituminous roads, of which 2 miles were in Massachusetts and the other 16 in Ohio. Except for the oiled earth roads, of which more than 2 500 miles (included with the gravel and other low-type improvements in the previous classification) had been built in California by the so-called petrolithic process, these 18 miles were the only representatives, in 1904, of a type which was destined shortly to become exceedingly important. Sheet and rock asphalt surfaces had been laid in the cities since the Seventies, and both bituminous macadam and bituminous concrete surfaces had been built in England before 1890, but the only bituminous surfaces to be found anywhere in the United States in 1904 were 2 miles of tarred road in the Town of Tisbury, Mass., and 13 miles of bituminous macadam and 3 miles of asphalt in Ohio.

Modern Progress in Its First Decade

This year—1904—marks the end of a period. To that time there had been no important change in the methods of road construction that had been used for a century or more. Either of the major types of surfacing—gravel and macadam—was known to give entire satisfaction under the traffic normal to the country roads of the time. The other types that had been

developed and used in small mileage, such as the shell roads of the Tidewater States and the sand-clay roads of the South, were suggested by the availability of the materials rather than by any difference in the demands of the traffic that used them.

Viewed broadly the few types of surface constructed up to this time may all be considered as of one class. In the construction of all, the same principles governed; in all, a fragmental mass was bound together more or less firmly by a natural cement in the manner made familiar by a century of practice; and all alike depended for their efficiency on the conic principle of pressure transmission by which they spread the vehicular loads and thus reduced the intensity of pressure borne by the sub-grade.

That need was felt for no other kind of construction was due, of course, to the fact that the traffic on all roads was much the same. Even in the more populous States the greater part of the traffic using the roads consisted of relatively light, horse-drawn, steel-tired vehicles, to which were added near the cities a bicycle traffic which, although it might attain considerable volume, was never more than a negligible factor in determining the type of surface. This was the normal traffic condition which existed practically up to 1904. What makes that year a turning point in highway history is the fact that about that time there began the great outpouring of motor vehicles from the cities, which quickly set the inter-city roads apart from others as a class requiring different treatment.

The peculiar effect of the automobile on water-bound macadam roads is so well known as to require no description, and the manner in which the road builders met the challenge by substituting tars and asphalts for the weaker mineral binders has been an oft-told tale. First as dust layers, then as protective surface coatings, then as binders, introduced into roads of the macadam type by penetration, and, finally, as hot admixtures according to the bituminous concrete principle, these materials, borrowed from the stock in trade of the city street builder, solved the automobile problem in a manner that was apparently entirely satisfactory.

The effect of this development in the road-building art is shown by comparison of the statistics of 1904 and 1914, the dates which, to all intents and purposes, mark the beginning and crest of the wave of bituminous construction. In 1904, according to the records, there were in the entire country only 18 miles of bituminous rural roads, all in the two States of Massachusetts and Ohio. By 1914 there were 10 500 miles, a mileage which was nearly three-quarters of the aggregate length of all roads of higher type than macadam. This was the high-water mark of the lower forms of the bituminous types. That it by no means marked the end of their usefulness is indicated by the fact that 3 367 miles of the surface-treated and penetration types were built in 1924. The recession of the tide is indicated, however, by the fact that the mileage of the two types existing in 1924 was less than 50% of the mileage of all types better than water-bound macadam in comparison with the 75% level reached in 1914.

It is generally recognized that these two types, which came into use with the development of passenger automobile traffic, are especially adapted to that class of traffic. The relative decline in their use began when motor trucks in considerable numbers began to appear on the rural highways; and coincidentally an increasing swing toward the rigid pavements of concrete and brick and bituminous concrete on a concrete base is noted. The turning point was reached in 1914, or perhaps a year or two earlier.

Although the first concrete road had been built in 1893, there were no more than 5 miles of that type on rural highways in the entire country in 1909. In that year approximately 4 miles were built; in 1910, about 20 miles were added; the following year, 40 miles; and then the first big increase occurred in 1912, when more than 250 miles of rural highways were paved, to be followed in 1913 with 500 miles and in 1914 with more than 1 500 miles. At the close of 1914 there were 2 348 miles in the entire country; and 10 years later the mileage had increased to 31 146 and construction was proceeding at the rate of more than 6 000 miles per year, a rate approached by no other type better than gravel.

The more extensive use of brick, and of bituminous pavements of the mixed type on concrete base, began also at about the same time and was due to the same cause—the increased use of motor trucks. In 1914 there were approximately 1 600 miles of brick pavement; in 1924, there were 4 319 miles. In 1914 the mileage of rural highways paved with bituminous concrete or sheet asphalt was still negligible; in 1924, there were more than 9 700 miles of these types.

Trunk Lines versus Farm-to-Market Roads

The decade following 1904 was marked not only by the development of new types of road, but also by two other changes of even greater significance. The first of these was a general increase in the radius of travel by highway occasioned by the use of the automobile; and the second—the natural result of the first—was a change in the character of the public demand for highway improvement.

In 1904, the automobile had still to prove its ability for sustained performance. Its ownership was still limited to a small and wealthy class. The popular demand for improved roads was, therefore, still predicated on the use of the bicycle and the horse-drawn vehicle. The farmers, always conservative, were still, for the most part, either actively hostile to road improvement or lukewarm in support of it. In general, their demand was for the improvement of the roads connecting their farms with the railroad shipping points or near-by towns. More positive influence was exerted by city and town merchants who sought by road improvement to extend the trading radius and business of their towns, and by the limited but influential class of motorists who longed for smoother, mud-and-dust-free roads on which to operate their vehicles. All these influences combined at first to produce a demand for short stretches of improved roads radiating from the towns and rail shipping points. Later, as the automobile was perfected and its users

became more numerous, the latter created a demand for longer, unbroken stretches of improved roads, forming a network connecting the larger towns, a claim that was resisted by the farmers who continued to favor the so-called farm-to-market type of movement.

In the smaller Eastern States the conflict never became acute, largely because the distance between towns and market points was so short that the farm-to-market plan of improvement, when carried to its ultimate development, became practically identical with the inter-town or trunk-line plan. Thus, the issue was satisfactorily settled in Rhode Island as early as 1902 by the adoption of a definite system of State highways for construction by the State Board of Public Roads. A similar proposal by the Highway Commissioner of Connecticut, made originally in 1906, was enacted into law by the State Legislature in 1913; and, in the meantime, Maryland had settled the question definitely by the adoption of an inter-county seat, trunk-line system to be improved and maintained in its entirety with State funds under the State Roads Commission. Maryland's system was designated in 1908, and was the first to be placed completely under State control for both construction and maintenance.

That the controversy was not so quickly settled in many of the other States was due mainly to two reasons: First, the important lines of travel in a number of the States were not sharply defined. In some this resulted from sparsity of settlement, and in others from the contrary condition of close settlement with numerous centers of more or less uniform size and importance. States such as Texas and Wyoming were typical of the first group. In them, the long distances between centers and the condition of the roads delayed the development of highway traffic between the towns and promoted a use of the highways largely as feeders to the rail lines; and the same remoteness of the towns one from another prevented the early harmonizing of the two plans of development as in the smaller Eastern States by the evolution of one into the other. Of the second class there were such States as Iowa, Kansas, and Wisconsin, in which the very number and uniform size of the town centers caused a diffusion of traffic over many roads and delayed the recognition of routes of outstanding importance. In these States, also, the towns are essentially agricultural centers, and this fact contributed further strength to the demand for farm-to-market roads as opposed to trunk lines.

The Federal Aid Road Act

The instances mentioned furnish examples of one of the reasons for the prolongation of the controversy which raged over the question of farm-to-market *versus* trunk-line development. The second reason was simply that many of the States as yet had no State agency for the administration of a highway plan of State-wide scope, and the development of the trunk-line plan naturally presupposes the existence of such an agency.

The second of these reasons was promptly removed after the passage of the Federal Aid Road Act in 1916 by the provision of that Act requiring the

creation of adequate highway departments in all States as a condition precedent to participation in the benefits of the Federal aid.

The Act of 1916 appropriated $75 000 000 to be expended in five years under the direction of the Secretary of Agriculture in co-operation with the State highway departments, for the improvement of post roads. It provided for the apportionment of the appropriation among the several States in proportion to their area, population, and mileage of post roads; and it permitted the expenditure of Federal funds for improvement in any amount up to 50% of the cost, but not exceeding $10 000 per mile. This latter limit was later raised to $20 000 and then reduced to $15 000 as at present.

Since 1916, amendments have been made to the Federal Aid Road Act appropriating, or authorizing the appropriation of, additional funds to a total, including the fiscal year, 1927, of $690 000 000; and the mileage of road improved under the co-operative plan up to June 30, 1926, amounted to nearly 56 000.

A first step toward the ultimate settlement of the trunk-line question in all States was made when the U. S. Bureau of Public Roads, acting for the Secretary of Agriculture, as one of its first administrative acts requested of all States the submission of a 5-year program map showing the system of roads on which the State highway departments would request Federal aid during the period covered by the appropriations provided by the first Act of Congress. Although the systems designated in response to this request were understood to be merely tentative, the request of the Bureau had the effect of directing attention—in many States for the first time—to the desirability of establishing a definite program for the improvement of a system of highways as distinguished from the more or less casual improvement of unrelated sections of roads.

Load Limitation a Consequence of the World War

The Federal-aid work had scarcely begun, however, when the World War intervened and practically put a stop to all operations; and the war did a number of other things to the existing improved roads which, however disastrous they may have appeared at the time, have turned out to be blessings in disguise. At the outset the construction and maintenance of highways were declared to constitute a non-essential industry. As a consequence new construction, except as required for the immediate service of the Army, was greatly curtailed. Maintenance also was greatly hampered by the difficulty of obtaining the necessary materials and the scarcity and high wages of labor. At the same time there was released upon roads generally inadequate to stand it an unpredecented traffic of heavy motor trucks. To this experience and the heavy damage which followed, the public owes the development of most of the sound principles and policies which now govern the improvement of highways.

The first result was a strong reaction against the use of heavy motor trucks. There were large numbers of people who, forgetting that a road is of service only in so far as it accommodates the need for economical transporta-

tion, demanded that the manufacture and operation of vehicles too heavy for the existing roads be prohibited. As few of the roads were designed to carry motor-truck traffic, to have taken this course would have amounted to the throttling of a new development in transportation before it had a chance to demonstrate its utility, and it was rightly opposed with great energy by the manufacturers of motor vehicles. The latter, on the other hand, took a position at the opposite extreme from which they demanded the right to manufacture and sell vehicles of large capacity and heavy weight, without regard to the strength of the roads, on the theory that the greater the capacity of the vehicle the smaller would be the cost of operation per unit of capacity. Their slogan was, "build the roads to carry loads", and this was met by the opposite party with the equally dogmatic demand that the loads should be limited to the capacity of the existing roads.

The issue thus joined, the principals to the controversy—highway officials on the one side and the manufacturers on the other—wisely agreed to submit their differences to the test of mutual discussion; and out of the series of conferences that ensued there came an agreement on certain fundamental facts and principles, which has served as the basis for a harmonious co-operation of the two groups, and which now constitutes the foundations of highway improvement policy in all States.

It was agreed at the outset that, for the first time in history, the weight of vehicles had become a critical factor in rural highway design. Hitherto, the minimum practical thickness of road metal had been sufficient to carry the maximum vehicular load. The development of the motor truck had altered this situation. It called for stronger surfaces that would spread its heavier load over a wider area of the sub-grade in order to reduce the intensity of the pressure to an amount that the soil could support.

It was clear also that whereas deterioration of the highways had previously resulted mainly from the attrition of the surface, a new form of deterioration approaching rapid destruction would result unless the roads on which the heavier motor trucks were being operated were strengthened so as to enable them to carry the increased weights; and whereas, the amount of the deterioration had formerly been a function of the volume of the traffic and of time, the new destruction by excessive weight might be caused by a few vehicles in a very short time.

It was agreed, therefore, that the highway officials must have definite knowledge of the maximum weight to be supported as a first condition of design; and this knowledge was supplied, in a measure, by the voluntary decision of the manufacturers to limit to 7½ tons capacity the future production of vehicles. Engineers were thus assured that if, in the reconstruction of the thoroughfares on which heavy trucking had developed, they would design to accommodate a vehicle of 7½ tons capacity they would not see their handiwork quickly destroyed by vehicles of much greater size and weight.

Differences Between Main Highways and Local Roads Accentuated

This alone, however, was not a sufficient basis for the design of all roads. The building of roads of sufficient strength to carry 7½-ton trucks required

a heavy investment of public funds, which could be justified only if the economies inherent in the transportation of goods in vehicles of large capacity were sufficient to outweigh the increased cost of the roads. It was recognized clearly for the first time that the cost of highway transportation is made up of the cost of the highways and the cost of operating the vehicles over the highways, and it was agreed that the common purpose of the public highway officials, vehicle manufacturers, and operators should be to reduce the total cost of transportation rather than one or the other of the elemental costs.

It could be proved that the number of trucks of large capacity already using some of the highways—principally those radiating from and connecting the larger cities—had already grown to the point where the combined savings in operating cost would more than balance the greater cost of providing highway service for them. As to these highways there could be little doubt of the wisdom and economy of building a type of surface adequate for the heavy truck traffic. Other roads, similarly located with respect to cities, had not yet developed a sufficient amount of the heavy traffic to repay the additional cost of the stronger construction, but it was not difficult to foresee that such a condition would develop in the future. On the majority of the roads, however, the development of traffic of sufficient weight to justify the higher types of construction was very remote; and it was apparent that the one-time prevailing condition of uniformity of traffic on all roads had been definitely broken down. Instead, a new and much different condition had arisen under which the main inter-city roads were found to be carrying traffic far in excess of the much greater mileage of local roads.

Under the new condition the economic justification for the improvement of the main roads lay to a far greater extent than formerly in the reduction of transportation costs and to a lesser degree in the effect on the value of property. The main roads had become through traffic arteries, as distinguished from the more numerous local roads, which continued to be of value primarily through the service they rendered in giving access to the land.

As to the main roads, which carried a wide-ranging traffic, it was now clearly apparent that the character of their improvement must be commensurate with the density of their traffic; that continuity of improvement was of the highest importance; and that the traffic was already so great that the loss in operation of vehicles in the absence of road improvement would exceed the cost of improvement. These roads also were distinguished in one other respect, namely, that their traffic tended to increase far more rapidly than that to be found on the local roads, the condition of which remained much as it had been. Where the main roads carried long-distance traffic, the local roads served the traffic of a neighborhood; where the main roads were collectors of traffic, the local roads were feeders and distributors; where the traffic of the main roads tended to grow in direct proportion to the growing use of motor vehicles and the growing resort of industry and the entire people to highway transportation, the local roads served the much lighter, and, from the standpoint of growth, the far more stable traffic produced by a single agricultural community.

It became apparent, therefore, that the economic justification of local road improvement would continue to rest largely in the value and importance of the land; that, in the main, the traffic would demand only a low type of improvement; and that continuity of the improvement was not so essential as in the case of the main, through roads.

The need of continuity in the improvement of the main roads was the first of the new conditions to be met with appropriate action. From 1915, all States in rapid succession designated systems of State roads, including generally the main inter-city roads, to be improved under the more or less direct supervision of the State highway departments; and the several State systems were substantially welded into a National network by the designation, in accordance with the Federal Highway Act of 1921, of the Federal-aid highway system on which it is now required that all Federal appropriations shall be expended.

Economic Principles Enunciated

Continuity of improvement of the main roads thus being assured, it remained for a Joint Committee representing the American Association of State Highway Officials and the National Automobile Chamber of Commerce to enunciate a policy with respect to the rate and manner of the improvement, which could win general support and adoption.

Briefly, that policy may be stated as follows: It is accepted as a truism that the volume of traffic on the main roads is so great that the economies in transportation effected by road improvement clearly outweigh the cost of the improvement. This being true, the improvement should proceed as rapidly as available supplies of labor and material will permit and without other limit. All roads should be improved to the degree justified by the operating savings that may be expected to accrue to the traffic, and no road should be improved to any greater degree. Where the mileage of road to be improved is so great that the type of improvement indicated by the traffic can not be completed on the whole mileage within a short period the most important sections should be raised immediately to ultimate type, and the remainder of the mileage should be advanced through the initial stages of grading, draining, and low-type surfacing, in order to spread as much of the benefit of improvement as quickly as possible over the entire road system, further improvement to await the completion of the first stage over the whole system. This is known as stage construction, and it is the only feasible practice in the numerous States in which a large mileage of main roads remains to be improved in the face of a traffic already highly developed. It is also the logical plan of development for the main roads of the States in which traffic has not yet grown to the proportions justifying high type surfacing.

In any case the stage-construction plan takes account of the rapid growth of traffic, which is a characteristic especially of the main roads, by providing fully in the initial stage for the subsequent construction. Grades and alignment are designed to meet ultimate requirements; drainage structures are built of durable materials; rights of way of ample width for the future are

obtained; and the initial surfacing becomes the sub-base of the second-stage surfacing. Obviously the soundness of the plan is contingent on the complete and continuous maintenance of each stage of the construction, a kind of maintenance which—thanks to the war experience and the standard established by the Federal Highway Act—practically all States are now prepared to give.

The accepted policy contemplates the improvement of the main roads, to which these methods are applicable, as a responsibility of the States to be assumed through the agency of the State highway departments, and financed, in large measure, by the revenues derived from the taxation of vehicles and motor fuel. The local roads are viewed as the responsibility of the counties and lesser subdivisions. With a few important exceptions, as in the vicinity of large cities, the degree of improvement required does not rise above the lower types of surfacing, the expense of which may be met, as it should be, by taxation of the local land and property.

These, then, are the outstanding developments in highway improvement of the post-war period: The classification of highways according to traffic density; the designation of State highway systems in all States, the systems including the heavy-traffic highways of State-wide importance; the interconnection of the State systems by means of the Federal Aid System; the improvement of roads in accordance with traffic demands to the limit set by probable operating savings; the stage-construction plan of progressive improvement of the entire system; and the development of adequate maintenance provisions. In the main, all are outgrowths of the war experience fostered by Federal aid.

Highway Research

One other great advance has characterized this period—the application of scientific research to the problem of developing types of construction and methods of administration and finance adequate to meet the demands of the fast-growing traffic. In this, also, the initial impulse came from the Federal Government, and, in co-operation with State highway departments and universities, it is continuing to support numerous studies in several fields, as a result of which there is being built up gradually the structure of a new science—the science of highway engineering.

The investigations include studies of the characteristics of materials—sand, stone, gravel, bituminous materials, cement, concrete, and brick; determination of the forces applied to road surfaces by standing and moving vehicles; of stresses developed in the structure of roads and bridges by live loads, by temperature, and by other natural causes; analyses of sub-grade soils and tests of methods designed for their improvement; and studies of the flow of water through drainage structures, of the run-off from drainage areas, of the effect of moisture on soils, and many others of fundamental importance and value.

Popular interest has centered on the large scale tests, such as those of the Bates Road, for which entire credit is due the Illinois Highway Department; the Pittsburgh, Calif., experiments; the impact tests at Arlington, Va.; and

the intensive studies of highway traffic conducted by the U. S. Bureau of Public Roads in co-operation with the authorities of Connecticut, Maine, Pennsylvania, Ohio, California, Tennessee, and Cook County, Illinois.

Much that is of immediate practical benefit has already been derived from these investigations; but, for the most part, they are dedicated to the future. It is not to be expected that their fullest benefits shall be realized immediately. The building of a science is a laborious, a painstaking process, and at present the groundwork is not much further advanced than were the foundations of the modern science of medicine and surgery fifty years ago. If fifty years hence the science of highway engineering has been built up to the point now attained by the physicians and surgeons, this effort now being put forth will be abundantly repaid, and not too late. For the improvement of highways in the United States is a process that must be continued indefinitely.

Highway Conditions at the Close of the Second Decade

At the end of 1924, according to reports received by the U. S. Bureau of Public Roads, there was a total of 3 004 311 miles of highway in the United States, of which 707 138 miles had been improved in some degree, 235 471 miles by grading and draining, and 471 667 miles by surfacing. Of the roads surfaced, 64 086 miles was improved with sand clay, 246 357 miles with gravel, 60 467 miles with water-bound macadam, 45 172 miles with bituminous macadam, 9 691 miles with bituminous concrete, 31 188 miles with Portland cement concrete, 4 710 miles with brick and other block pavements, and 9 996 miles with various unclassified types of surface.

It is idle to talk of completion when less than one-fourth of the 3 000 000 miles of highways has been graded, only one-sixth has been surfaced, and one-sixtieth paved; when little more than one-half the mileage of the main State roads has been improved with any kind of surfacing, and there remain on these important arteries thousands of unsubstantial one-way bridges and dangerous railroad grade crossings; when the number of motor vehicles registered is doubling every fifth year and the traffic with them; when the size of cities, the magnitude of industries, and the amount of material wealth are increasing at an almost unprecedented rate. As long as these conditions continue it will be necessary to continue to build, maintain, and rebuild the roads.

At the present rate approximately 40 000 miles per year is being surfaced and the annual expenditure approximates $1 000 000 000. There is no indication of an early reduction in these rates of construction or expenditure, dwarfed as they are by the annual production of a 1 000-mile procession of motor vehicles and an annual expenditure for operation approaching $10 000 000 000. The nation has set its hand to the economic improvement of means of highway transportation. It is not a task to be accomplished in a day. It is, and must be, a continuous process. There is only one limit which may reasonably be set. It is this: No road should be improved by expenditures of public funds in excess of its earning capacity. The return to the public in the form of economic transportation is the sole measure and justification of the degree of highway improvement.

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THE QUESTION OF ROADS IN FRANCE*

By P. Le Gavrian,† Esq.

Synopsis

The traveler who to-day examines the roads of France, and compares them with those of other well-civilized countries, is struck by two facts: On the one hand, the fine arrangement, the excellence of the routes and the completeness of the plan on which most of the great roads have been established, and the splendid lines of great trees which shade them and call attention to their presence afar off; and, on the other hand, the moderate, too frequently even, sad condition of their surfaces.

This fine system evokes the idea of a strong organism, well constituted and strongly built, but whose muscles have been spent, for a time, by prolonged under-nourishment to a state of inferiority in comparison with the function which one would have the right to demand of them.

What does it mean? Modern traffic, crowded, exacting, and more and more numerous, leaves the great roads whenever it is able, and seeks in the secondary system of parish roads, the route, less direct perhaps, but retaining to a greater degree that smoothness which the motor vehicle claims to-day—more smooth until the parish road, worn out, spent, full of holes, even to the foundations, also becomes impracticable for one's tour.

France has the good fortune to possess in its system of national roads, a network of fine highways, anciently built, generally well laid out, covering all the territory with their meshes.

By adding some parish roads, judiciously chosen, and establishing upon the system as constituted a regimen of rational upkeep, properly manned and equipped, and sufficiently provided with necessary financial resources,

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one could endow the country with an economic tool of first class, an instrument of exchange by highway for touring, commercial, and industrial purposes. Whether one wishes it or not, transportation by highway is growing as an extension of railroad service, and, frequently, as a parallel or substitutionary service, and is one of the most active elements of National prosperity.

And one must not forget, either, the essential rôle the roads are called on to play in the organization of National defense. The World War—a war of transport if any war ever was—has shown clearly services of first order which a good highway system can render in defensive (defense of Verdun) and in offensive warfare (Battle of the Somme, final offensive of 1918).

The different aspects of this important question of the highways of France, therefore, are discussed in this paper.

Historical Account

It was not without difficulty and without a long series of efforts and of changes that the system which France has to-day was evolved through the centuries, and it must not be allowed to deteriorate.

The Romans when they conquered Gaul, and established the first highways of communication, methodically built the Roman roads, which are found there. They are generally laid out in a straight line, paved with large flagstones, and sometimes with a kind of macadam similar to modern macadam.

Abandoned toward the end of the imperial rule, they were, after the invasion of the barbarians, the object of various attempts to restore them or to create new ones, especially under the Merovingian kings and under Charlemagne, great leader of "hosts." One finds in many localities, especially in the northern and northeastern sections of France and Belgium, long lines, occasionally distinguished by rows of trees, or suggesting at times traces of rough roads which, on the maps "d'Etat-Major", are designated "Chaussée Brunehaut."

In the Middle Ages, travel and transportation were limited except by horseback and by donkey, and travelers were satisfied with mere trails. Feudalism neglected the roads; nevertheless, Philippe-Auguste for the first time paved the two principal roads which pass through Paris, from north to south and from east to west.

In the Thirteenth Century, there appeared the first attempt at organized transportation—the privileged messengers of the University of Paris. This service was later improved by Louis XI, with the institution of "Postes," but the purpose of this improvement was chiefly political.

In 1283, Beaumanoir distinguished five kinds of roads: Path, trail, track, road, and royal road, the latter 64 ft. wide.

Under Charles VIII, came the introduction of coaches and carriages. Until then, journeys were made only by horse or litter.

With the last Valois kings, in the Sixteenth Century, the royal power tended more and more to separate itself from the feudal order and to become

centralized; still more clearly one finds among the preoccupations of the Government that of securing communication throughout the kingdom; in fact, the requirements of administration and authority necessitated more frequent relations between the ruler and his subjects.

Francis I allotted the proceeds from the tolls for the upkeep of great roads. Henry II began the regular paving of roads with the Paris-Orleans Highway. In 1552 Charles Etienne published a "Guide to Great Roads" in which are named and defined routes already representing a length of 25 000 km.

Under Charles IX the first public coaches were run. Under the Bourbons the order and authority of the royal power, which increased more and more, demanded still greater progress.

In 1599 Henry IV created the "Charge de Grand Voyer", which he confided to Sully, an edict of 1607 defining its duties. This is the first Highway Code, in which is manifested the royal will to remove obstacles and hindrances to traffic.

Temporarily suppressed by Louis XIII in 1621, the office of "Grand Voyer" had to be re-established in 1645, in order to stop the continual encroachments to which the great roads were exposed. In 1661, the accession of Colbert to the "Controle Générale des Finances" gave a new impulse to the development of the public ways; he committed the administration to special commissioners and the roads received, thereafter, grants of royal assistance.

Nevertheless, aside from the roads which joined the "Maisons royales" with each other and with the capital for communication with the capital, travel through the territory as a whole remained dangerous, precarious, and difficult. Memoirs of the times which relate the travels of persons of quality throughout the kingdom, abound with accounts of incidents and adventures, coach accidents, etc.

It is from the reign of Louis XV that one must date the actual organization of the highways in France. In 1716, the "Corps des Ponts et Chaussées" was created with Ormesson, the two Trudaine (1743-69), Cotte (1777), Chaumont de Millière (1781), Perronnet, the famous designer of the "Pont de la Concorde" at Paris. The decrees of 1705 and 1720 ordered the roads laid out in "avenues" and it is to their application that France owes the fine order of the major part of the great highways of the National system.

Tresaguet, Engineer of the "Généralité de Limoges", formulated for the first time the exact rules for the construction and maintenance of macadamized roads; reviving, to a certain extent, the Gallic-Roman method, he established, as new in theory, the need of laying roads on a solid foundation, a principle which, later, was lost to view under the influence of the Englishman, MacAdam, and which modern constructors have again restored to honor, with just reason.

In 1776, the length of the great highways reached 40 000 km., and an edict proposed to classify them according to their length. Under the Consulate and the Empire, the organization of highways, temporarily disturbed by the

Revolution, adjusted itself. The Imperial Routes were divided into three classes and received normal appropriations from the public funds, the first two imposed entirely on the State; the third, half on the State, half on the Departments.

During the Restoration and under the Monarchy of July, the great routes (called then "Routes Royales") were returned to the State; but the attention of the public authorities was chiefly centered on the organization of the parish roads ("Petite Voirie"). The law of May 21, 1836, established the charter for this last class (classification of the various types of parish roads, financial management for their construction and maintenance, compulsory labor, subsidies, etc.). In 1851 and 1852, there intervened the law and decrees by the "Police du Roulage" which have recently been reformed in order to form the present "Code de la Route" (1922).

During the second half of the Nineteenth Century, however, the birth and development of railroads deprived the roads themselves of a large part of their importance. The great roads were neglected, and the 40 000 km. of highways (imperial, royal, or national, according to the political régime) which the public no longer required as formerly, saw even their existence put to question.

After 1870, they were in a deplorable state and public power once more aroused itself; the "Freycinet" program for great public works, formulated in 1878, applied to their reconstruction an appropriation of 150 000 000 francs, an allowance which, although very insufficient, has been realized only progressively during a long period.

On the other hand, the parish roads developed considerably, thanks to the financial aid that the State granted to the Departments and the Communes. Their favor is even such that there is thought of abolishing outright the national roads, in order to incorporate them into the parish. Happily, the Government, believing that this part of the National equipment may one day be recovered, opposes all attempts at transference of the national roads.

The appearance and rapid growth of the automobile came soon to give them reason in reversing the rôles and in returning to the great arteries their ancient importance, greatly increased.

Automobiles and Their Effect on Roads

The exact number of animal-drawn vehicles used on the roads and highways of France is not known; statistics for direct taxes show as subject to taxation: 1 600 000 carts and 1 000 000 animals. As, however, the tax reaches only certain classes and leaves free the mass of lumber wagons and industrial and agricultural vehicles and their horses, these figures represent only a fraction of the elements of transportation. It is not rash to suppose that the total of these may reach 3 600 000 vehicles and 2 400 000 draft animals.

With respect to automobiles, accurate figures are now available, as since 1920 all motor vehicles have been subject to taxation and consequently have been registered.

For 1924, the figures of the Minister of Finance indicate: 370 000 vehicles used for the transportation of persons; 200 000 vehicles used for the transportation of merchandise; 20 000 side-cars; and 30 000 cycles, making a total of 620 000 vehicles, representing a force of 6 500 000 h.p. These figures, representing an advance over those of former years, are to-day greatly exceeded.

Admitting that the animal-drawn carts are still used mainly for short hauls and on the parish roads (around farms, villages, etc.), the great majority of automobiles go almost everywhere and circulate on roads of all types, by preference on those of the highest type. How do the various elements of transportation affect the surface of a roadway and damage it?

Effect of Weight.—A loaded vehicle, whatever may be its method of traction or propulsion, grinds the exposed stone over which it passes; the wheels tend also to dislodge and fracture the stones which form the road surface; and, finally, although these effects may not be immediately noticed, the result is apparent to the least attentive observer.

The remedy for this type of destructive action rests in the limitation of loads, or at least in the limitation of the unit pressure that the tire transmits to the soil.

The "Code de la Route" prescribes that in any case this pressure shall not exceed 150 kg. per cm. of width of tire, this width being measured in contact with a hard surface on a new tire (hard or elastic) in a state of normal action.

Tangential Effects.—It is well known that in an automobile the power of the motor, transmitted through the main shaft and differential, is transferred by a gear which actuates the rear wheels; this gear, in order to cause the vehicle to move, must overcome the resistances opposed to the motion arising from the action of gravity, slope, air resistance, rolling resistance, etc.

The resultant of all the forces applied at the point of contact between the wheel and the road surface may be resolved into two forces, namely, the one perpendicular to the surface (normal component); the other parallel to it (tangential component).

The normal component, if the ground is not very sloping, conforms sensitively to the weight that the wheel transmits to the surface; for a given vehicle and a given load it does not vary.

The tangential component, on the contrary, has very different values according to the speed of travel and especially according to the positive and negative accelerations that come unexpectedly into this velocity, depending on the condition of the surface, the nature of the tire, etc.

Calculations show that, in the case of a touring car weighing 1 500 kilos—distributed, two-thirds on the driving axle and one-third on the front axle—and mounted on pneumatic tires, it is necessary to apply a motive force of 32 kg. to maintain a uniform speed of 55 km. per hour over a macadam surface in good condition. The tangential component, T, is 6 kg. at the rim of each of the rear wheels.

If the vehicle approaches an incline of 3%, it is necessary, in order to maintain the same speed, that the force shall be increased from 32 to 77 kg.; the tangential component, T, increases from 6 to 28.5 kg.; that is, it is nearly quintupled. In order to increase the speed and accelerate in a second from 55 to 58.5 km., it must, in this period, increase to 173 kg.; T increases then correspondingly to 79 kg.

This simple example shows that the tangential component, T, is not considerably influenced by the speed, if that remains uniform, but that the intervention of a slope increases it appreciably, and that it is especially increased—and in considerable proportions—as soon as an acceleration is communicated to the vehicle.

These are, therefore, the disturbances, even unimportant ones, from the normal system of travel, which bring a sudden increase of the destructive effects exercised by vehicles on the highways.

These calculations, however, do not take account of the disturbing elements arising from the inequality of the rolling surface; in this case the tangential forces assume still higher values, combined as they are with the impact, oscillations, and bounds of the vehicle, and with effects of friction and the slipping of tires.

All these forces, applied to the exposed surface of a road, shake, dislodge, and disperse, first the finest elements, then the middle ingredients, producing small holes and slight cracks; and, finally, the larger elements are attacked in their turn, and the holes and cracks are enlarged and dug out. Thus are created the pot-holes, then the ruts, which increase and, by spreading, endanger the existence of the road. The destruction of ordinary macadam invariably follows the process indicated; it is accompanied by a considerable breaking up into dust.

At the beginning of the automobile era, a great effort was expended to combat this dust, primarily to avoid the unpleasantness; but it was very quickly perceived that the entire road was being broken down and disappearing into dust and that in preventing the formation of the latter, the former would be preserved.

When one has ascertained the processes of destruction of highways under traffic, one has to set to work to discover the remedies.

The Remedies.—It is legitimate to demand in the construction of vehicles that they shall satisfy certain conditions, for example, that the loads shall be proportioned to the width of the rims; that speed shall be limited according to the weight of the vehicle; and that tires, springs, shock absorbers, etc., shall be improved. It is more necessary still to construct the road in accordance with the destructive forces which it is called on to undergo, since, in fact, the roads are made for the vehicles and not the vehicles for the roads.

What does that imply? To lay out the roads in accordance with an engineering plan and, where an existing road is concerned, to correct and adjust its construction so that vehicles may be operated with ease, without requiring of the motors abnormal tractive effort or too sudden variations of acceleration; to study the turns with a view to reducing the centrifugal forces

and skidding; to clear the view; to adjust the width of the road in proportion to the intensity of the traffic, etc.

Especially is it necessary, in addition, to construct the roads with smooth surfaces, without depressions or bumps, of well-bound materials, the elements of which properly solidified will be neither easily crushed by the load nor easily displaced and dislodged by the rear wheels; to diminish thus, by all possible means, the abnormal development of tangential forces and shocks.

It is, in fact, a conclusion which must not be lost sight of, that the formation of irregularities in the rolling surface is as much the cause of all the misconceptions that assail the highways as the material of which it is made. The interests of the highway and the interests of the vehicle are by no means antagonistic; on the contrary, they are the same.

Finally, it is necessary to regulate and improve the use of the road system by establishing suitable traffic rules and by marking the public way with proper direction and warning signs, clear and legible, which will serve to direct the traveler and enable him to approach with safety the "Points singulières" of the route, if such exist.

The study of the various problems concerned with the administration of roads and the practical solutions that have been attempted during the past fifteen years in all civilized countries, have been carried on and discussed periodically by technical men and road users through the International Road Congress.*

Various Methods of Surfacing.—From the point of view of highway surfacing, especially, great progress has been made. It is recognized that the ordinary macadam (water-bound) is no longer suitable except for minor roads or roads with light traffic; but various compositions have been tried, and it can be stated that the engineer provides at present a scale of processes, the number of which is increased each day, varying in efficacy and correspondingly in net cost.

First come the simple surface treatments (tarring, bituminous, and emulsions) which, spread on the macadam road, offer a protective covering, sufficient for a traffic of swift and light vehicles.

Again, the binding of the road materials may be accomplished by the introduction of a fluid material into the interstices of the broken stone (penetration method), or by a previous coating of the stone (mixing method). The binding or cementing material may be a bitumen (tar or asphalt), lime, cement, silicate of soda, etc.

There are produced in this way, "Pitchmacadam" and "Tarmacadam", mortars and bituminous concrete (of types "monolastic", "bitulithe", "Trinidad", etc.); mortars and asphaltic concrete (types "Duraphalt", "Asphalte-Porphyre", etc.); roads of cement concrete (ordinary concretes or special concretes, "Soliditit", "Rhoubénite", etc.); and silicated roads. These various

* The International Road Congress has its headquarters at Paris, 1, Avenue d'Iena. It unites official representatives from 40 Governments, includes more than 1 000 adherents, and comprises large administrations (Departments, cities, chambers of commerce), sporting and technical associations (automobile clubs and touring clubs) and professionals ("Chambres Syndicles d'Entrepreneur de Constructeurs d'Automobiles," etc.), of numerous countries. Five large congresses have already been held: Paris (1908); Brussels (1910); London (1913); Seville (1923); Milan (September, 1926).

surfaces withstand heavy and intense traffic all the better when the composition and bond are dense and firm.

Finally, there are pavings of regular elements manufactured in advance: Stone pavings may be in small mosaic elements, or in the form of blocks which constitute the most solid and resisting surface; wood, asphalt, concrete pavings, even highways of rubber, seem destined especially for city traffic, or for special use (highway bridges, for example).

Management of a Heavy Traffic System in France

The simple definition of the various methods of surfacing with their proper qualities shows that the rational management of a highway system should admit of a methodical distribution of their use. The kind and intensity of traffic vary a great deal from one region to another—industrial and heavy here; touring and light there; mixed elsewhere. Climate plays its rôle, and is important; humid regions ought not to be treated in the same way as dry regions. Topography also has its effect causing the elimination, on steep slopes, of certain surfaces reputed to be slippery, which, on the contrary, would be sought in a flat country.

Finally, and above all, considerations of expense impose the necessity of strict discrimination. Modern surfaces are expensive; it is not sufficient, in order to solve the problem, to choose the best and the most solid; one would give a great deal if the merciless question of money could be avoided and such a conception could be realized.

The management of a large road system should observe these many conditions, adapt solutions to the needs, and should especially adopt a progressive attitude and adapt itself to continuous improvement. Consider, therefore, to what a program of management for the French system would amount.

At present, since the return of the Provinces of Alsace and Lorraine to France, the system includes, in round numbers, 40 000 km. of national roads; 13 000 km. of departmental roads; and 650 000 km. of parish roads of various kinds, to which must be added the rural roads and city streets, the statistics of which are not available.

The national roads are entirely under the direction of the State and are placed under the authority of the Minister of Public Works. Departmental roads are in charge of the Departments, but they tend to disappear, the majority of the Departments adopting the legal alternative of classing them among parish roads.

Parish roads divide themselves into main highways ("chemins de grande communication") and common roads ("chemins d'intérêt commun"), administered by the prefects (parish service) for the Departments and the Communes, and into ordinary parish roads directed by the mayors at the expense of the Communes.

There are, then, from the point of view of administration and budgeting, two well-marked classes: The national roads, centralized in the hands of the Government and maintained by the budget of the State; and the parish roads,

which, in accordance with the plan of each Department, are dealt with by the local authorities and budgets.

Evidently, there is no such systematic grouping of the various roads on the basis of the intensity, nature, and importance of the traffic, as there is in the administrative classification. Nevertheless, it may be assumed without reasonable error that all the national roads, with few exceptions, and a fraction of the departmental roads, and the main highways ("chemins de grande communication") receive in reality the principal stress of traffic, especially from the through traffic, or that which is not purely local.

One may consider, therefore, as separate from the mass of roads which traverse the country, and as constituting a network of principal traffic arteries, almost the entire system of national roads, completed by some heavy-traffic routes of the parish highways. A bill formulated by the Government with a view to fixing the conditions of such a classification and providing for the necessary maintenance of these routes, has actually been submitted to the vote of Parliament. Fully respecting the ownership of the parish roads, and without affecting the departmental rights, its object is to make possible the application to these "grand traffic routes" of methods of management consistent with a program exceeding the limits of the Department, and of putting these roads in harmony with the remainder of the principal traffic system, the State contributing to the expense.

Of the roads, 7 000 or 8 000 km. could be usefully thus classed and added to the national highways, in order to form a "reseau de circulation principale" from 45 000 to 50 000 km. in extent.

How will this principal system be treated? One is led immediately to divide the technical problem, and, consequently, the financial problem which is intimately connected with it into two main divisions including, respectively:

(*a*) Improvement and adaptation (construction operations), which can be undertaken only gradually, section by section, following a methodical program.

(*b*) Maintenance, properly speaking, consisting in maintaining a sufficiently good condition on all roads of the system at the time, either in their present state, or in the condition which they would attain as the result of their progressive improvement.

The writer can discuss only from memory this general maintenance which is very insufficiently assured to-day, due to the lack of money, but which will be possible without technical or administrative difficulty when the highway program is provided with suitable funds on which the service will be able regularly to count, the amount of which appropriation for the 50 000 km. of the primary system should be fixed at 400 000 000 francs annually (under the present condition of traffic).

As for the operations of maintenance and betterments (correction of lines, width and grades, curves, new surfaces for roads, etc.), it will be expedient to regulate the arrangement, the method for securing the improvement of routes being taken in order, without scattering efforts. Fig. 1 shows the primary highway system to be reconditioned.

By way of example, the writer conceives that in the main system, previously defined, the first work to be executed over many years, would be the improvement of such great roads as:

Paris-Amiens (about 125 km.), with extension from Amiens to Lille and Dunkerque (180 km.), and from Amiens to Boulogne and Calais (125 km.).

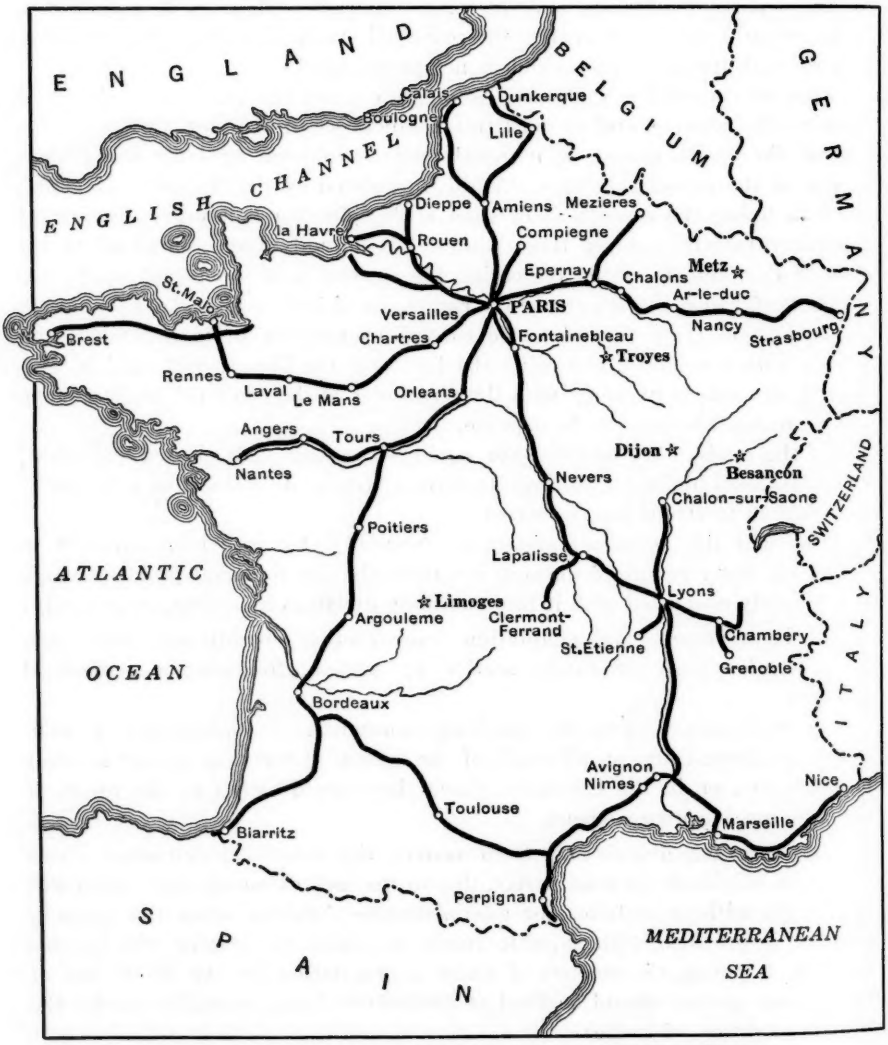

FIG. 1.—MAP OF FRANCE, SHOWING PRIMARY HIGHWAY SYSTEM.

Paris-Rouen-Le Havre (175 km.), and Rouen-Dieppe (55 km.).

Paris-Evreux-Caen (200 km.), with branch to Lisieux-Trouville (25 km.).

Paris-Chartres-Le Mans-Laval-Rennes-St. Malo (400 km.) and across toward the coast, Granville-Dinan-St. Brieuc-Brest (250 km.).

Paris-Orleans-Blois-Tours-Angers-Nantes (415 km.).
Tours-Poitiers-Angoulème-Bordeaux (325 km.).
Bordeaux-Bayonne-Biarritz-Hendaye (200 km.).
Bordeaux-Agen-Toulouse-Carcassonne-Narbonne (350 km.).
Perpignan-Narbonne-Montpellier-Nîmes-Avignon (230 km.).
Paris-Fontainebleau (through Corbeil and Melun) Briare-Nevers-Moulins-Lyon (500 km.), with branch from Vichy and Clermont-Ferrand (75 km.).
Lyon-St. Etienne (65 km.).
Lyon-Grenoble (100 km.), with branch from Chambéry and Aix-les-Bains (50 km.).
Lyon-Valence-Avignon-Aix-Marseille (300 km.).
Marseille-Toulon-St. Raphael-Cannes-Nice-Mentone (200 km.).
Paris-Meaux-Chalons-sur-Marne-Nancy-Strasbourg (450 km.), with branch from Epernay-Reims-Mézières (100 km.).
Paris-Compiégne (70 km.).

These roads, as well as the approaches to some important cities (Fig. 1), which it would be desirable to improve (such as, Metz, Limoges, Dijon, Besancon, Troyes, etc.), represent a total length of 5 500 km. in round numbers. Also, there should be included in the program the special work involved in the building of two great mountain highways, known as the "Route des Alpes" and "Route des Pyrenees."

The improvement of some of the routes in question has already begun. Thus, Paris-Amiens is resurfaced with a bituminous surface as far as Chantilly; Paris-Rouen and Paris-Caen have a common bituminous trunk from Paris to Nantes, where they divide, toward Evreux and toward Rouen, two branches improved with the same kind of surfacing; Paris-Orleans has undergone a similar transformation, finished in 1925; Paris-Chartres is bituminous as far as Rambouillet, etc.

What improvements are necessary on these main roads? In order to fix one's ideas, consider the diagonal which crosses France from Calais to Nice by Amiens-Paris-Nevers-Lyon-Avignon and Marseille.

Under present conditions, it will be suitable to adopt a minimum width of 6 m. which should be increased to 7 m. in passing through villages, and to 8 m. or more in especially frequented sections (approaches to Paris, Lyon, Marseille, etc.).

As far as possible the route will avoid the congested cities. The curves will be improved in radius and in visibility, and systematically provided with super-elevation to permit vehicles to pass with safety and especially without cutting the corners on the wrong side. At cross-roads and junctions of important tributary roads, the intersection will be treated in such a manner as to obtain clear visibility as perfectly as possible (widening, cutting sides, removal of obstacles), and the signaling will be perfected. Finally, and above all, the roads will be adapted to the traffic; and this adaption will involve an extensive plan, a detailed study of the right surfacing for each section of the route; for example (extensive plan):

From Calais to Amiens, roads of the tar macadam or penetration macadam type, and surface-treated roads.

From Amiens to Paris, bituminous or asphalt roads replaced in swampy or wet sections with roads of cement concrete.

The cross-roads, at present paved by the localities, will be kept up in paving (or mosaic as the case may be) or changed to concrete.

In the suburbs of Paris, there is a stone paving which will be improved or replaced with concrete.

Beyond the limits of the Parisian suburbs, as far as Briare, bituminous or asphaltic roads or macadam with surface treatment.

From Briare to Nevers, Moulins and Lapalisse, macadam roads with surface treatment.

Branches toward Vichy, bituminous or asphalt or tar macadam roads.

From Lapalisse to Lyon, surface-treated macadam roads, replaced at the approach to Lyon by concrete or mosaic paving, the same to be used in the southern suburbs of Lyon.

From Lyon to Avignon, use tar macadam (or similar) and surface-treated macadam with substitution, as the case may be, of asphalt and bituminous surfaces or block paving at the approaches to or through cities (Valence, Avignon, etc.).

From Avignon to Aix, tar macadam (or similar) and surface-treated macadam.

From Aix to Marseille, tar macadam or asphalt or bituminous roads, paving with stone in the suburbs of Marseille.

From Marseille to Toulon and to St. Raphael, paving with stone or concrete (suburbs), tar macadam, asphalt, and bituminous surfaces and surface-treated macadam following the sections.

From St. Raphael to Cannes, to Nice and Mentone, asphalt and bituminous surfaces with concrete at important crossings.

In the general outline, thus traced for each route, the plans would fix in detail the choice of material to be used under the various conditions of soil, climate, and type of traffic.

Financial Means

It has already been stated that the work of execution, under what has been called the "Reseau Principal," should include two parts:

(1) Maintenance, properly so-called, which would necessarily be continuous, and which has been estimated at the annual cost of 400 000 000 francs (in the present state of traffic, the figure being susceptible, consequently, of increase in the future if the traffic on the highway increases as is probable).

(2) "Constructive work," that is, the improvements and alterations of routes, by successive sections in proportion to the resources which could be allotted.

What financial means can be resorted to in order to meet these needs? The first conception is that of the creation of an Office of Highways. The idea, started some years ago by the "Conseil Superieur des Travaux Publics," received at once the support of the Touring Club of France and the majority of the Chamber of Commerce. The principle was as follows: An official organization, but endowed by civil, personal, and financial resources, and able, consequently, to obtain funds, to make loans, and being withdrawn from the annual uncertainty of the French budget system, capable of pursuing continuously, effectively, and rapidly, the realization of a program of work liberally conceived.

This office would be placed under the authority of the Minister of Public Works and administered by a "Conseil" appointed by the latter, which could include among its members not only technical men (engineers), but also representatives of the users of the highways.

Such an institution functioned in England for fourteen years; created under the name of the "Road Board", it has become to-day the Road Department under a Director General (connected with the Ministry of Transport), and has given extremely satisfactory results.

As for the financial resources assigned to the office, they would come naturally from taxation, but from taxation rationally distributed. Various systems have been proposed, in this regard, either the allotment to the Office of all fees and taxes whatsoever, collected as tolls for the use of the roads (rights of passage, occupation of soil or subsoil, etc.), and from traffic (duties on horses and carriages, taxes on automobiles, gasoline, etc.), or the allotment of only part of such taxes.

A plan of legislation, laid aside by the Government of 1923, has since been taken up, revised, and laid aside again by the Bureau of the Chamber of Deputies (January 5, 1925) and reported very favorably before the Commission of Public Works by M. le Deputé Charlot; but no result has followed, because it is contrary to the principle of budget unity and conflicts on this account with the opposition, until now impregnable.

Aroused by the extension of the situation which became more aggravated each day, the Minister of Public Works then decided to consider another system. On the one hand, he asked from Parliament—in the budget for 1926—a substantial increase of the normal credit for maintenance of national roads;* and, on the other hand, he proposed with the consent of his colleague, the Minister of Finance, a new plan of legislation supporting the execution of a program of construction and reconstruction. This plan would distribute over five years the execution of the program, which, reduced to the strictest economy, would allow an expenditure of 50 000 000 francs.

As a result of this beginning by the Minister of Public Works, however, a group of Deputies, on their part, prepared a resolution which was introduced on June 29, 1926, before the Bureau de la Chambre. This resolution, signed by Mm. Charlot, Le Trocquer, A. Borrel, Sibille, etc., was inspired by the idea expressed by the Minister of Public Works, but it completes and extends it. Herewith is a brief analysis:

The program would include the reconditioning of about 10 000 km. chosen from the most worn-out roads in the amounts of: 500 km. of paved roads; 300 km. of special surfaces; and 9 000 km. of surface-treated macadam.

The roads restored would be about four-fifths national roads and about one-fifth departmental, recognized as having an intensive traffic requiring their assimilation into the national roads.

The expenditure would be distributed thus:

First year	90 000 000 francs
Second year	120 000 000 "
Third year	150 000 000 "
Fourth year	180 000 000 "
Fifth year	210 000 000 "
Total	750 000 000 francs

* The figure voted in the budget of 1926 has been increased to 312 000 000 francs.

In order to cover these yearly expenditures, the Minister of Finance would be authorized:

(*a*) To collect during five years an exceptional road surtax on all vehicles.

(*b*) To retain during five years, one-half the increase above that reported for the fiscal year 1926, of the proceeds from the internal gasoline and fuel taxes.

(*c*) To complete the amount required in case of insufficient annuity, by an exceptional subsidy on the budget of the State.

Such is at present the situation, and such are the needs with which those in charge of the highways of France are confronted. It is important that the solution shall be quickly arrived at because the longer it waits, the more it will cost.

In any case one thing is certain, that is, that a considerable financial effort ought to be made for the highway system, and that it is justified, whatever may be the economic difficulties of post-war conditions, because it concerns one of the vital elements of National equipment: communication and transportation by highway, which of all forms of communication is the most considerable in volume, the most important in tonnage, and reaches the greatest number of inhabitants of a country.

The writer would like, in this paper, to silence a plausible argument which one hears occasionally maintained. Why make vehicles, which are operated only in sections where there are perhaps no national roads or roads of heavy traffic, support the expenses of improving the latter? First, because the system of main roads is sufficiently extensive that all may use it to a greater or less degree; next, because the condition of highways giving access to a region interests the region as a whole; and, finally and especially, because if the great arteries are not in a condition to fulfill their office, the traffic which normally makes use of them avoids them and is diverted over the regional highways, to the detriment of their preservation, the depletion of local finances and to the great inconvenience of local traffic.

Conclusions

The great road system of France, created and developed through the centuries, and one of the most remarkable in the world, is wasting away as a result of a prolonged under-maintenance, coincident with a heavier and heavier duty. The technical services which have charge are doing all they can, but are unable to cope with the destruction, which increases each day and which financial resources, long insufficient, will not compensate.

These roads are as indispensable to the economic life of the country as the railroads, canals, telegraph, and electricity. From the point of view of engineering art, remedies exist; it is enough to make use of them; the writer has enumerated them and has shown in what spirit it would be expedient in his opinion to apply them; but the engineers must turn to the financiers, to whom they must look for the great sums which the roads imperiously demand, if they are to be restored and enabled to perform the rôle which they are called on to play in modern life.

HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

A SYMPOSIUM*

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Note.—Written discussion on this paper will be closed in **January, 1928.**

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HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

INTRODUCTION

By Harrison P. Eddy,* M. Am. Soc. C. E.

One hundred and fifty years ago the inhabited part of the United States consisted principally of the area east of the Allegheny Mountains and between the St. Lawrence River and the Spanish Province of Florida. The first Census of 1790 for the new Republic showed a population slightly less than 4 000 000.

Westward migration began during the Revolution and has continued with the extension of boundaries. Naturally, the course of migration was along the river valleys where ease of traveling was obtainable. Towns sprang up around border trading posts and forts such as Pittsburgh, Cincinnati, Detroit, St. Louis, and Chicago.

The living conditions of the great mass of the people were extremely simple. Houses were of logs or of loose unpainted clapboards. Cooking was before open fires or in brick ovens. Lighting by night was by tallow candles, the sperm-oil lamp, or by the open wood fire. The friction match had not been invented, and flints and steel and tinder were the means for procuring fire. Clothing was chiefly homespun manufactured in the household with spinning wheels and hand looms. Few public water supplies and no public sewerage systems existed. Plumbing was practically unknown. Primitive methods for the removal and disposal of household waste and excretal matters prevailed.

At the time of the first census, only four municipalities had populations in excess of 10 000—New York, N. Y., 33 131; Philadelphia, Pa., 28 522; Boston, Mass., 18 320; and Baltimore, Md., 13 503. The remaining population was living under rural or semi-rural conditions.

The development of water supply and water purification, of sewerage systems and sewage treatment, and of refuse collection and disposal, has been coincident with the growth of cities. Land drainage has followed westward migration and the establishment of farms in the level areas of the western river valleys. Until the end of the Nineteenth Century the part played by the mosquito in the transmission of malaria and yellow fever was unknown, but, in the last two or three decades, an extraordinarily successful campaign has been waged against the mosquitoes conveying these diseases. In the early days the problem of ventilation was non-existent but, to-day, with large gatherings of individuals in halls, schools, and theatres, the problem has become acute.

During the 150-year period there has been an unprecedented utilization of science and engineering for the amelioration and control of man's environment. The papers which follow, outline the progress in those branches of engineering which have done so much to promote human health and comfort.

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

WATER-WORKS

By George W. Fuller,* M. Am. Soc. C. E.

Prior to 1870 the history of water-works in the United States presents a picture of isolated and spasmodic community efforts to obtain supplies, then, for about a generation, increased activity due largely to the enterprise of business men making water-works equipment, and then a continuous and fairly active growth caused by the rapid increase in urban population, culminating in the general recognition—since 1900—that the design and operation of modern water-works offer opportunities for research, engineering, and administrative skill of the highest grade.

The few water-works of the Colonies were crude affairs, frequently consisting of a bored-log pipe line leading from a masonry-walled hillside spring to a trough in the center of the town, as exemplified by the plant built at Schaefferstown, Pa., in 1730. As a rule, however, these early community works were large wells, from which the householders carried water, as is done in little European hamlets to-day. According to Mr. M. N. Baker, there were 17 water-works in the United States at the close of 1800, all but one privately owned. It seems probable that public wells used by communities would increase this number, but the now unknown instances of such works can be discovered only as a by-product of more important historical research, through old newspapers and local records, in diaries, and in books of travel.

The early Seventies was the period of the great fires in Chicago, Ill., and Boston, Mass., and there has been a tendency to attribute the increased rate of water-works construction during this decade, from 243 plants in 1870 to 598 in 1880, to a better appreciation of the importance of public water-works as a fire-protection resource and an increased standard of living that demanded running water in homes. The facts, however, seem to be that the most important influences were furnished by promoters working for manufacturers of water-works equipment. The late J. James R. Croes, Past-President, Am. Soc. C. E., explained† the situation as follows:

"The rapid increase in the number of water works constructed within the last sixteen years is mainly due to the competition of enterprising business men. About the year 1870 Mr. Birdsill Holly, of Lockport, N. Y., a builder of pumping engines, began an active canvass of the towns which had no water supply, with the view of furnishing a type of engine and pump manufactured by him by which the pressure was pumped directly into the mains without the use of a reservoir, and at any desired pressure variable at will of the engineer, thus enabling the whole pipe system to be used as a fire engine if necessary. By organizing a private corporation to build works and asking from the public only a certain fixed annual subsidy based on the number of fire hydrants furnished and guaranteed to deliver on demand a fire stream under 100 pounds pressure, the Holly Company were able to induce many towns to allow water-works to be built.

* Cons. Engr. (Fuller & McClintock), New York, N. Y.

† In the preface to the third (1887) edition of his "Statistical Tables of American Water Works."

"The practicability of the scheme being proved, and the financial success of several such enterprises having been demonstrated, other pump builders followed the example of the pioneer, and with such success that competition has compelled all parties to build better pumping machinery than ever before and to increase their guarantees of efficiency, durability and economy of operation."

Table 1 shows the available data on growth and ownership of water-works used for public supply in the United States. These data are taken from the "Manual on American Water Works Practice," published in 1925, by the American Water Works Association. When it is realized that more than five years' work, with the help of more than 300 collaborators, resulted in a volume of more than 800 printed pages to tell what is good water-works practice, it will be appreciated how meager must be the historic outline which can be set forth in the limited space here available.

TABLE 1.—GROWTH OF WATER-WORKS IN THE UNITED STATES, AND CHANGES IN CHARACTER OF THEIR OWNERSHIP.

Year.	Total plants.	Public plants.	Private plants.	Ownership unknown.	Percentage of Total.		
					Public.	Private.	Unknown.
1800	16*	1	15*	....	6.3*	93.7*	....
1810	26	5	21	....	19.2	80.8	....
1820	30	5	25	....	16.6	83.4	....
1830	44	9	35	....	20.5	79.5	....
1840	64	23	41	....	35.9	64.1	....
1850	83	33	50	....	39.7	60.3	....
1860	136	57	79	....	41.9	58.1	....
1870	243	116	127	....	47.7	52.3	....
1880	598	293	305	....	49.0	51.0	....
1890	1 878	806	1 072	....	42.9	57.1	....
1896	3 196	1 690	1 489	17	52.9	46.6	0.5
1924†	9 850†	6 900†	2 950†	....	70.0	30.0	....

* Since this table was compiled, another private water-works, built prior to 1800, has been discovered.

† Estimated.

Two supplementary comments are needed, to indicate:

First, that although the proportion of privately-owned water companies is less than previously, the hit-or-miss type of franchise, or contract both as to quality of service and rates of charges, has largely disappeared. In its place have come State regulatory commissions which, under the provisions of the police powers of the State, now supervise rates of charges and in many States the quality of service, with or without the co-operation of State health commissions.

Second, the water-works industry is more extensive than is indicated by the number of plants used wholly or in part in supplying water to the public. The water-works for the exclusive use of many of the large industrial plants are both extensive and important. Water for condensing purposes is highly important, although the subject is clearly outside the scope of this paper.

Distributing Systems

There is little positive information about the distributing systems leading from the primitive collecting works in American communities. In Great Britain, where it was customary to supply water to each consumer for only a short time during each day, cisterns were constructed in the attics of buildings and connected by lead pipes to the street mains. A man (called a "turncock") went around every day, turned a cock on the service pipe and filled the cistern. If it did not hold enough water to meet the requirements for the next 24 hours, the occupants either did without water or procured some from outside the building.

The scheme of intermittent supply never gained a foothold in the United States, but house cisterns were used, on account of low street pressures. They served a useful purpose, during periods of scanty supply, by making available, during the day hours of heavy draft and consequent low pressure, water which slowly filled them during the night.

The early pipes were practically all bored logs, sometimes with a ring of iron shrunk on each end to prevent splitting. Such pipes had long been used in Europe, and were not a makeshift adopted on account of the exigencies of construction in a new country, as is sometimes stated. As a matter of fact, bored wood pipe are still made and used to a considerable extent, chiefly for mine drainage in the East and for water-works on the Pacific Coast. The manufacture of such pipe, and of banded stave pipe was greatly improved about 75 years ago by Wyckoff, of Elmira, N. Y. The banded type was the forerunner of the large banded wood-stave aqueduct pipes used to-day in the West for irrigation, hydro-electric plants, and water-works.

About 125 years ago cast-iron pipe began to supplant wood pipe in England. Such pipe, with flanged joints, had been tried by the Chelsea Water Company, of London, about 1746, but without much success on account of difficulty with the joints. After studying the causes of the trouble, Thomas Simpson, the Company's engineer, about 1785, designed the first successful bell and spigot pipe with a lead joint. It was quickly adopted, but the facilities for manufacture restricted its introduction for some time. Such pipe, imported from Great Britain in 1807 and laid in Philadelphia, inaugurated the use of cast-iron water mains in the United States.

It was soon found, however, that, in some cities, the interior of uncoated pipe carrying water became tuberculated, and that the carrying capacity was much inferior to that of new pipe. The problem of preventing tuberculation was solved by Dr. Angus Smith, who in 1848 produced the coating that has since borne his name. Although this tar product, which rapidly came into general use, did not prevent tuberculation entirely, it was an excellent and inexpensive deterrent when made of the materials and prepared and applied as he recommended. After the success of this coating was proved, cast-iron pipe had no serious rival for water-works, except in localities where transportation charges made it preferable to use materials which were less expensive although they might not be so durable.

Quite recently a desire for a still better coating for cast-iron pipe, has developed, and the subject is again one of the live topics of discussion in water-works circles.

As the demand for cast-iron pipe began to increase, there was a corresponding increase in the range of thicknesses adopted for pipe walls, and in the shapes of the bells, spigots, and special castings. This complexity at last became intolerable, and in 1902 a standard set of specifications for pipe and specials was adopted by the New England Water Works Association. In 1908 the specifications of the American Water Works Association were adopted, and have since been the standard in America. Although they may need revision to-day, as to coatings and quality of material, and perhaps in other respects, they have been invaluable alike to pipe purchasers and makers.

Wrought-iron pipe appeared in the Forties as a material for distribution mains and service pipes. The pipe was used as a shell, being lined with cement mortar and coated on the outside with some form of preservative. These cement-lined pipes had a decided vogue, particularly in New England, for about 25 years, but finally were judged less economical than cast-iron pipe in the long run. Owing to recent changes in commercial conditions, they are again back on the market.

In the Fifties, when hydraulic mining in California was becoming a notable industry, large quantities of water had to be dealt with. Although open flumes served as conduits in most places, there were occasional dips below the hydraulic grade where pipe of some sort had to be provided. Cast iron was out of the question on account of cost and transportation difficulties; wrought iron had to be used. When the late Hermann F. A. Schussler, M. Am. Soc. C. E., began the construction of the San Francisco Water-Works, in the Sixties, his knowledge of the satisfactory service of wrought-iron pipe in mining led him to use it for the larger mains of those works. As a consequence of the early satisfactory experience in San Francisco, wrought-iron and steel water pipe gained a foothold on the Pacific Coast from which they have never been dislodged.

The first important riveted wrought-iron pipe in the East was laid in Rochester, N. Y., in 1873-75, but it was not until after steel had replaced wrought iron to a great extent that the construction of riveted pipe was considered seriously in the East. Exterior protective coating and the lining of riveted and similar plate pipes have recently received much attention, and some of these methods are so encouraging that, even in the East, steel has become a strong competitor of cast iron in the larger sizes. Both materials, however, are relying, more than ever before, on coatings and linings to preserve their inherent advantages.

In this commercial struggle, active competition is arising from reinforced concrete pipe, for places where the pressure is not too great. It has been used for aqueducts and other pressure lines for little more than a decade, but in this brief period has shown encouraging results as to low first cost, freedom from leakage, and maintenance of discharging capacity.

Not until 40 or 50 years ago was any general attention paid to what may be called distribution economics. The early distribution systems, after a time, did not deliver the theoretical quantity of water they should, at different parts of a city. This seems to have been one of the reasons for the attention paid to service reservoirs and stand-pipes long before any serious study was made of the hydraulics of distribution systems as a whole.

In some cities attempts were made to improve the distribution of water by partly closing the gates of mains that fed districts where the pressure was highest. When serious fires made an ample supply necessary in a section fed by one of these throttled mains, it was found that the practice was a hazardous makeshift. Engineers finally undertook a careful study of the subject, and an early result was a paper, presented in 1892 to the New England Water Works Association by John R. Freeman, Past-President, Am. Soc. C. E., which attracted wide-spread atttention.

Many years prior to this, one cause of low street pressures was known to be the leakage of water through defective plumbing fixtures, services, and street mains. The leakage inside buildings was attacked by metering the services, but progress was very slow for years, except in some of the thrifty New England cities. Where metering was impracticable, attempts were made to inspect the plumbing, but this also met with violent objection in many cities. To-day, it is difficult for anybody unfamiliar with the facts to realize the bitterness of the opposition to metering as late as the Eighties, and it is fitting in this general discussion to mention the long and forceful campaign for metering conducted a generation ago by Henry C. Meyer, F. Am. Soc. C. E., in his paper* and in other ways.

The opposition to metering service pipes on dwellings was so strong that the method of detecting waste, by dividing a city into districts and then searching for all instances of leakage in one district after another, was developed. In its most elaborate form, due to Mr. G. F. Deacon, of the Liverpool Water-Works, the water delivered to each district was measured by a continuously recording meter. By inspection of plumbing and by using early forms of what is now called the aquaphone, most of the serious leaks and cases of waste in a district were detected and remedied. After that, the district meter register indicated any material increase in waste. The system was tried in only one city of the United States—Boston. It was successful technically, but was so expensive that it was finally abandoned.

When Clemens Herschel, Past-President, Am. Soc. C. E., invented the Venturi meter, in the Eighties, he furnished the first reliable device for measuring large quantities of water supplied through pipes from reservoirs and pumping stations; for the actual discharge of pumps in those days was affected to such an extent by the unknown slip in the pump valves that the only sure thing about the discharge was that it was much less than the reading of the pump counters. Then, in the late Nineties, E. S. Cole, M. Am. Soc. C. E., developed the pitometer, by which the discharge of any pipe can

* *The Sanitary Engineer.*

be determined. With these devices, everything formerly accomplished by the Deacon district system can be done at much less cost.

Obtaining Water Supplies—Sources

With the early water-works there was rarely any difficulty in finding near at hand ample supplies which seemed to be of satisfactory quality. In Philadelphia, for example, after the early wells were deemed inadequate, an unlimited supply could be obtained by pumping from the Schuylkill River.

In New York, the case was unusual. Public water-works began with public wells, the first dug by the Dutch in 1658. Many others were dug later, and those in each ward were placed in charge of its alderman and councilmen. The wells in the heart of the town became so polluted and the water so offensive that it was little used. Just before the Revolution, a private company, under the direction of Christopher Colles, sunk a large well near the present site of the Tombs Prison, from which a less objectionable supply was pumped, by a Newcomen engine, to a small reservoir and then delivered by gravity through log pipes. The enterprise was soon abandoned on account of the war.

Next came Aaron Burr's water-works, built by the Manhattan Company in 1799, under an Act of the Legislature. The works consisted of a large well, two pumps, and the reservoir built by Colles. The Company made hardly more than a nominal attempt to furnish water, as its real object was to conduct a bank, permitted by an innocent looking clause, in its perpetual charter, regarding the investment of its surplus funds. In 1823, it had 23 miles of pipe, nearly all wood, two small steam pumps, and a masonry reservoir holding 550 000 gal. This plant and many wells furnishing water of very inferior quality supplied the city until 1830, when a well, 16 ft. in diameter and 112 ft. deep, was sunk at 13th Street and Broadway, most of it in rock. The water was too hard for domestic purposes, and was used chiefly for fire protection. A small steam pump raised the water to a large cast-iron tank, and the supply was delivered through about 9 miles of cast-iron pipe, from 6 to 12 in. in diameter. This was the first municipal plant in New York.

By 1832 the supply furnished in the city by the Manhattan Company and by the public and private wells was so offensive that all who could afford to do so purchased water brought in hogsheads from unpolluted wells in the northern part of the island. About 600 hogsheads were furnished daily in this way at $1.25 per hogshead. A very bad outbreak of cholera during that year was followed by an increase in the already strong demand for an adequate public supply. It was at first thought that water could be obtained from the Bronx River, but later it was shown that the nearest available supply was from the Croton water-shed. In 1835 the Croton project was approved by the citizens at a special election, bids for construction were opened in 1837, and Croton water, practically the first fit to drink after the days of Dutch control, was delivered in 1842, when the city had attained a population of more than 300 000.

Intakes.—The construction of intakes, by which water was drawn from rivers and ponds, developed so slowly and by such small advances that it is hardly possible at this time to give credit for significant improvements in design, except in one respect. The first public water supply in Chicago was established in 1831; it was merely a public well. The municipal water-works were begun in 1851. After the sewerage system of the city was built and the sewage was carried out into Lake Michigan by the Chicago River, apprehension was felt that the water about the intake in the lake would become dangerously polluted. The extension of the intake pipe farther out into the lake, beyond the area of pollution, was the remedy most favored. The City Engineer, the late E. S. Chesbrough, Past-President, Am. Soc. C. E., opposed this vigorously and insisted that only a supply drawn through a tunnel in the lake bed would be free from serious danger of interruption of service. Such a subaqueous tunnel was then an engineering feat of the first order, but it was at last built in 1867, establishing a practice followed in many cities, not only on the Great Lakes, but also on some of the larger rivers.

Dams and Reservoirs.—Turning to those works for which a supply is obtained by impounding surface waters, one naturally recalls first the dams which form the reservoirs. The development of the details of gravity dams has been slow but continuous. The outstanding novelties have been of a different nature, and of rather limited availability. First there is the hydraulic-fill dam, a form of construction first used by the late James D. Schuyler, M. Am. Soc. C. E., in making railway embankments in Western Canada; then the rock-fill dam, with an impervious covering on the back; and the arched dam, particularly suited for high, narrow gorges. Somewhat later the hollow dam was introduced, for the purpose of saving part of the cost of gravity dams. It has developed into two types, both having transverse parallel buttresses supporting the up-stream back. In one type the back, of reinforced concrete slabs, slopes gently upward to the crest, and the weight of the water on it contributes to its stability. In the other type, nearly vertical arches between the buttresses form the back. Although only bare mention can be made of these types, the general interest in them has resulted in a large volume of printed information regarding their relative merits.

Hydrology.—One of those features of the development of water-works practice that has had a profound influence is inextricably tied up with the gradual increase in the knowledge of hydraulics and hydrology. For a long time so much water was available that there was no shortage except what was due to lack of ordinary common sense in procuring a supply. Little by little, however, there were shortages that could not be explained by the engineering knowledge of those times, and so water-works engineers began to search for their causes. Later, the necessities of irrigation and hydraulic power development increased the interest in this group of subjects, culminating quite recently in the interesting utilization of the mathematics of probability, under the leadership of Allen Hazen, M. Am. Soc. C. E.

So far as the co-ordination and direct utilization of this kind of knowledge is concerned, it should be mentioned that in the Nineties the late F. P. Stearns,

Past-President, Am. Soc. C. E., in investigations for the Massachusetts State Board of Health, and C. C. Vermeule, M. Am. Soc. C. E., in studies of the water supplies of New Jersey, for the Geological Survey of that State, developed general methods of adjusting reservoir capacities to the run-off from a water-shed. Just before that time there had been in the West, a few cases of expensive irrigation reservoirs, which could not be filled. The economical maximum capacity of reservoirs to utilize the run-off of a water-shed was, therefore, manifestly more than an academic topic.

Underground Water.—Underground water supplies are more numerous than surface supplies, although their total yield is not so great. It is unfortunate, therefore, that nobody has yet undertaken to gather and digest all the existing definite information regarding the engineering aspects of underground supplies, as has been done with surface supplies.

Pumping Water

The Keystone State not only holds the record for the first public water supply, but also for the first public supply furnished by pumping. In 1754 Hans Cristopher Christiansen, a Danish millwright, began the construction of the first water-works of Bethlehem after the general method already in use in several places in Europe. His pump, constructed of lignum-vitæ and driven by an over-shot water-wheel, raised water from a spring to tanks on the hillside above the village.

The first steam pumps in America were used for draining New Jersey mines. They were built in England, of the Newcomen type as improved by Smeaton. The first Newcomen pump used for town supply was erected in New York just before the Revolutionary War by a short-lived private company.

The first steam pumping plant in Pennsylvania was put in operation in 1801, in Philadelphia, where a low-lift pump, on the Schuykill River at the foot of Chestnut Street, raised the water to an open basin, from which it flowed by gravity through a conduit in Chestnut Street to Centre Square, now the site of the City Hall. There, at another station, the water was elevated about 50 ft. to wooden tanks, of about 17 000 gal. capacity, on top of the building.

The boilers at both plants were wooden boxes, 9 ft. high, 9 ft. wide, and 15 ft. long, of 5-in. white pine plank thoroughly bolted and braced. The fire-box, constructed of wrought-iron plates with vertical cast-iron flues, was inside. The engines were Watt's vertical, double-acting type of Cornish pump. The pump of each was of wood lined with copper. The steam cylinder was of cast iron.

It had been recognized, from the days of Savery and Newcomen, that the load on pumps is pre-eminently adapted for being carried by a steam engine. It is steady and always accompanied by enough water for condensing purposes. As a consequence, the early history of the development of the steam engine is largely that of the development of pumping engines. The condition is just the opposite, however, with boilers which furnish the steam to the pumps. The early pumping stations had boilers carrying very low pressure,

and, when more pumping capacity was added, the low pressure was retained in order that the old boilers could still be used. This condition kept station efficiency low, in spite of efficient pumping engines. As a matter of record of an important feature of pumping station development, the following statement, prepared by the late Charles A. Hague, M. Am. Soc. C. E., shows the maximum steam pressures in such stations:

Year	1800	1830	1850	1875	1900	1906
Steam pressure, in pounds.	5	20	50	75	125	175

There was a period of quiescence in the field of pumping machinery, down to the Sixties, due chiefly to patent conditions. Small pumps were usually direct-acting, and the larger pumps were improvements on the Bull-Cornish or Watt-Cornish types. In the Sixties the late Henry R. Worthington, M. Am. Soc. C. E., brought out the duplex, direct-acting pump, and other inventors followed soon with other devices for utilizing the duplex principle. At about the same time the Holly quadruplex crank-and-flywheel pump was introduced, and thus began a business competition between the two types of pumping engines that resulted in great improvements in each and a substantial reduction in their cost.

In England another type of pumping engine was developing, the Simpson compound engine, first used there in 1848, and in the United States in Philadelphia in 1872. Others were erected in a number of cities of the United States, the largest plant being that of the West Side Pumping Station in Chicago.

A few pumping engines used in the United States at that time were designed by consulting engineers, among whom the late E. D. Leavitt, M. Am. Soc. C. E., was prominent, but nearly all were designed by the builders. Competition was severe. It was necessary, not only to meet constantly rising guaranties, but also to reduce manufacturing costs, and naturally the improvements were in details rather than in general types. Finally, the experience with the Simpson type seemed so certain to establish it as the single standard for handling large volumes of water that the Worthington high-duty attachment, for the duplex engine, was brought out, and the Holly-Gaskill crank-and-flywheel engine was designed to improve the economy of the Holly quadruplex, crank-and-flywheel type.

In the Eighties, the Reynolds triple-expansion pump was designed, the first being built at Milwaukee in 1886; this soon became the standard high-duty, large-capacity, American pumping engine.

By 1900, earlier experience and commercial competition had rather definitely established certain types of pumping engines for the different kinds of service, and those interested in the subject will find them described by Irving H. Reynolds, M. Am. Soc. C. E.*

About 1900 the centrifugal pump became a really strong candidate for favor for water-works use. It was largely the development of reliable steam turbines and reduction gears that established such pumps in the favor they now hold in water-works practice. Originally, such pumps were used only

* *Transactions*, Am. Soc. C. E., Vol. LIV (1905), Part D, p. 513.

for low-lift service, but the field has gradually widened as the methods of design have been improved, and electrically driven centrifugal pumps are now operated against very high heads. This development has brought about one interesting change in pumping-station practice.

In the early plants these turbo-centrifugal units were generally placed in an old pumping station, where it was desired to handle the largest possible quantity of water with pumps occupying a definite space. The steam pressure was low, and other conditions were unfavorable for economy, but when turbo-centrifugal pumps were put in service under better conditions, using superheated steam at pressures most suitable for turbines, their economy rose very favorably, just as it did in the early days of turbines in industrial power plants and central stations. It became very evident then that station economy, rather than prime-mover economy, was the real measure of the success of the design.

Electrically operated centrifugal pumps, when the motors and pumps are suitably designed for the loads carried, are now standardized equipment. The chief drawback to their use is the possible unreliability of the current supply from outside sources. This is a serious matter where only a small quantity of water is stored in tanks or reservoirs.

The danger of interruption of current supply for motor-operated pumps, both centrifugal and reciprocating, has been met in many small pumping stations by adding a high-speed gasoline engine to help out in emergencies. Although the gasoline engine is cheap, its fuel is much more expensive than the oil used in Diesel engines. These were introduced about 30 years ago, and have been improved steadily ever since, until to-day they are capable of satisfactory operation for a considerable range of speed, although, like all internal combustion engines, they must not be given overloads. Two considerations seem to be chiefly influential in determining their use. The first is the present and probable future price of fuel oil and its availability at any place; the second is the length of satisfactory service of these engines. Concerning the second point, there is difference of opinion which probably cannot be reconciled until after longer experience with them.

The reciprocating well pump has been slowly improved, but, usually, it is not economical. The shocks and vibrations attending its operation and its relatively complicated construction make its maintenance high in comparison with the quantity of water it delivers. Attempts to avoid these drawbacks have taken two forms, both successful and widely used: The air lift and the centrifugal well pump, which deserve more than this brief mention.

Quality of Supplies

Presumably, this branch of the topic is the most important to develop in historic outline at present. Its consideration relates to the development of the modern germ theory of disease, particularly regarding the transmission of water-borne disease. In a strict technical sense, modern views as to the relation between the quality of a water and the health of the communities using such water date from the early or middle Eighties. Such matters,

however, were then under discussion for the most part by highly trained scientists in a few of the larger cities, and the general mass of water consumers, even including those well informed on educational topics in general, did not give much recognition to the significance of water-borne diseases until after the terrific outbreak of Asiatic cholera in Hamburg, Germany, in 1892-93.

The best index of the successful fight waged against water-borne disease in America is the decrease in the typhoid fever death rate, shown by Table 2. Although the pasteurization of milk, fly suppression, and the education of the public in the rules of sanitation and hygiene have contributed to this decrease, the improvement in the quality of water supplies has been the chief cause.

Prior to the development of modern views as to transmission of disease, there was a well-recognized belief that a public water supply should not be obtained from a filthy source. This theory, originating in England, had much to do in explaining the progressive changes made by numerous cities and towns in efforts to get water from clean sources. Most of the rivers of Western and Central Europe are only of moderate turbidity, and can be clarified suitably on sand beds such as were first constructed at London in 1829. The idea of these sand beds spread quite rapidly to various places in England and on the Continent.

Kirkwood's Report.—In 1866-68, the late James G. Kirkwood, Past-President, Am. Soc. C. E., then Chief Engineer of the St. Louis, Mo., Water-Works, made a thorough-going study of the methods used in filtering European supplies. It was a most admirable report, and its merit was well recognized by its translation into German and its becoming a standard work of reference in several European countries. On account of the unusually heavy turbidity or muddiness of the local rivers at St. Louis, Mr. Kirkwood did not recommend sand beds for the local supply, and this report was followed by no activities in the adoption of European filtration practice other than the two small sand filter beds which he built for Poughkeepsie and Hudson, N. Y., in the early Seventies.

Conditions in the Early Nineties.—It is difficult for the younger engineers to realize the extent to which the United States suffered from the scourge of sewage-polluted waters, which caused severe epidemics and in many places continuous or endemic conditions as to typhoid fever. The explanation is simple enough when one considers the close proximity of sewer outlets and water-works intakes. No wonder typhoid fever records were frequently from ten to twenty times higher than ordinarily found at present. The scourge was particularly severe in the manufacturing cities of Massachusetts and in the steel districts of Western Pennsylvania, where many of the population paid little or no attention to warning notices against drinking public water supplies. One of the sad features of the Columbian Exposition at Chicago in 1893 was the number of visiting engineers who contracted typhoid fever while there.

TABLE 2.—ANNUAL TYPHOID FEVER DEATH RATES PER 100 000 POPULATION FOR SOME AMERICAN CITIES.

Year.	Philadelphia, Pa.	Chicago, Ill.	Milwaukee, Wis.	Detroit, Mich.	Cleveland, Ohio.	Buffalo, N. Y.	Boston, Mass.	New York, N. Y.	Manhattan and Bronx, New York, N. Y.	Jersey City, N. J.	Baltimore, Md.	Washington, D. C.	Pittsburgh, Pa.	Cincinnati, Ohio.	Louisville, Ky.	New Orleans, La.	Minneapolis, Minn.	St. Louis, Mo.	Kansas City, Mo.	San Francisco, Calif.
1880	58	34	37	....	44	....	42	25	....	24	59	54	135	70	....	24	.....	40	.....	34
1881	74	105	47	....	99	67	56	38	....	63	58	46	151	71	....	30	.....	53	.....	37
1882	73	82	32	....	68	69	57	32	....	121	47	66	158	55	....	33	.....	45	.....	61
1883	64	62	25	....	65	34	52	32	....	49	35	62	107	51	....	23	.....	41	.....	64
1884	71	56	31	....	60	42	56	29	....	84	42	76	70	56	....	25	.....	42	.....	58
1885	64	75	28	....	34	23	40	27	....	71	42	65	76	42	....	17	.....	31	.....	44
1886	64	69	30	39	56	28	34	26	....	60	39	71	68	54	....	13	.....	30	.....	49
1887	63	50	31	65	52	34	44	26	....	54	41	78	128	142	....	15	.....	28	.....	23
1888	78	47	42	46	47	29	40	23	....	74	42	87	87	70	....	19	.....	31	.....	33
1889	71	48	28	31	74	30	43	24	....	83	47	94	95	49	....	17	.....	33	.....	55
1890	64	92	41	19	69	40	35	22	23	98	59	112	132	69	....	21	41	31	52	44
1891	64	174	33	34	50	51	34	23	25	100	35	82	101	62	....	24	55	29	42	42
1892	40	124	30	94	59	34	29	22	25	72	44	89	100	40	....	20	44	93	33	32
1893	41	54§	35	42	52	37	31	21	23	66	49	86	111	44	....	15	76	44	39	32
1894	33	38	25	28	29	59	29	17	19	53	47	91	56	55	....	29	56	34	26	37
1895	40	38	26	24	35	26	33	17	18	95	36	86	78	39	....	44	48	21	27	34
1896	34	53	18	23	48	20	31	16	16	84	38	58	62	52	....	33	32	20	23	26
1897	33	29	12	15	23	20	33	16	16	20	37	48	64	32	....	52	78	23	28	19
1898	46	41	17	22	34	27	34	21	20	40	36	71	73	33	....	66	44	17	37	41
1899	75	27	17	13	32	24	30	16	15	19	29	73	111	37	....	55	36	23	36	22
1900	35	20‡	17	27	54	26	26	21	18	21	35	78	144	39	64	40	39	29	40	13
1901	34	29	22	21	36	27	25	20	19	16	27	60	125	55	46	48	59	34	44	22
1902	44¶	45	15	24	33	33	24	21	18	20	42	78	134	62	61	45	27	37	36	27
1903	70	32	17	20	114	35	20	17	16	15	36	48	134	42	61	39	42	47	74	24
1904	53	20	14	20	48	23	23	17	13	19	37	47	142	79	63	36	42	36	39	27
1905	48	17	23	21	15§	23	20	16	13	19	36	48	99	40	51	32	25	20‖	53	24

TABLE 2—(*Continued.*)

Year.	Philadelphia, Pa.	Chicago, Ill.	Milwaukee, Wis.	Detroit, Mich.	Cleveland, Ohio.	Buffalo, N. Y.	Boston, Mass.	New York, N. Y.	Manhattan and Bronx, New York, N. Y.	Jersey City, N. J.	Baltimore, Md.	Washington, D. C.	Pittsburgh, Pa.	Cincinnati, Ohio.	Louisville, Ky.	New Orleans, La.	Minneapolis, Minn.	St. Louis, Mo.	Kansas City, Mo.	San Francisco, Calif.
1906	74	19	31	24	20	23	20	15	15	20	35	50*	130	70	71	30	34	17	32	58
1907	61	18	26	35	19	28	10	17	16	14	41	35	125	45	71	55	27	15	33	30
1908	35**	16	17	22	13	20	25	12	11	9§§	33	37	45	18*	47	33	19	14	29	19
1909	22††	13	23	23	13	23	14	12	12	9	25	33	23*	13	45	29*	21‖	16	30	14
1910	17†	14	45	20	19	20	12	12	11†	7	42†	23	28†	6†	32*	32	....	13	....	....
1911	14‡‡	11	20	15	14	25§	9	11	10	8	27§§	22	26	12	24	31	12	16	30§§	15
1912	13*	7	26	15	7†	12	8	10	7	8	24	22	13	8	22	14	12	11	12	14
1913	16§§ ‖	10†	12§§	25§§	14	15	9	7‡‡	7	11	22	16	20§§	7	24	17	12*	17§§	22	16
1914	8	7	8	11	8	16	9	7	6	8	21	11	15	7	27§§	22	13	12	16	13
1915	7	5	5	10	8	10§§	6	6	5	6	19	11	11	8	14	21	8	7*	10	9
1916	8	5	16	11	5	11	4	4	4	8	16*	12	9	3	14	23	6	10	12	3
1917	6	2§§	6	13	7	10	3	4	4	4	13	12	12	4	16	24	7	8	12	5
1918	5	2	6	8	5	7	3	4	4	5	10	11	10	5	15	20	9	8	15	4
1919	5	1	4	5	3	7	2	2	....	2	9	4†	4	3§§	6	13	3	6	11	3
1920	3.4	1.1	2.2	5.1	3.3	5.1	1.5	2.4	2.1	3.7	4.7	6.6	2.7	2.7	5.5	7.0	2	2.7	7.4	3.1
1921	2.2	1.1	1.9	5.8	3.5	3.5	3.2	2.4	2.1	3.7	5.5	6.4	3.2	3.7	4.7	9.3	1.3	3.8	11.5	4.0
1922	2.8	1.1	2.5	5.3	2.2	3.6	1.4	2.2	1.9	1.6	3.9	5.1	3.3	3.0	7.2	5.7	1.8	4.4	4.6	4.3
1923	1.5	1.9	0.8	4.0*	1.9	4.1	1.3	2.4	2.5	1.6	4.1	6.0	2.1	3.0	4.1	4.0	1.5	4.2	4.8	2.6†
1924	2.2	1.6	1.0	3.0	1.2	2.8	2.1	3.1§§	....	2.6	2.8	4.3	3.9	2.5	1.9	10.0	2.1	3.7	3.6	2.6
1925	2.3	1.5	1.4	2.7	1.5	4.5	3.5	3.3	....	4.1	3.6	5.0	3.2	4.2	5.8	19.8	3.3	3.9	1.9	2.2

* Filtration effective for city.
† Sterilization of water supply begun.
‡ Drainage canal in service.
§ New intake.
Coagulation introduced.
One per cent. of water supply filtered.

** Fifty per cent. of water supply filtered.
†† Ninety per cent. of water supply filtered.
‡‡ Sterilization of 66% of water supply.
§§ Sterilization of entire water supply.
‖ Coagulant applied during periods of high turbidity.

Lawrence Filter.—When the Massachusetts State Board of Health was re-organized in 1886, one of its most important steps was the establishment of the Experiment Station at Lawrence where, since that date, a vast fund of information has been developed on the subject of purifying water and sewage. One of the outstanding characteristics of these investigations, under the direction of the late Hiram F. Mills, Hon. M. Am. Soc. C. E., was that it embodied data of an engineering, chemical, and biological nature. Such thorough-going and unusually well co-ordinated data have had much to do with the development of sanitary engineering during the past forty years. In 1891-92, one of the most important pieces of work at Lawrence was an investigation of the ability of ordinary sand filters to remove typhoid fever germs, a matter of more than usual importance in the Merrimac Valley, where the Lawrence water-supply intake is only 8 miles below the Lowell sewers.

In the late summer of 1892 Asiatic cholera appeared on vessels quarantined in New York Harbor, and, under these circumstances, it was not difficult for Mr. Mills to obtain from the City Council of Lawrence, authorization for the construction of the Lawrence filter. This filter was completed in September, 1893, and embodied in its design the application of many of the data derived under Mr. Mills' direction at the State Experiment Station. The success of this filter, in reducing typhoid fever in that city, was so notable that it became a landmark in the train of developments which characterized the improvements in the quality of water supplies in America.

Louisville Investigations.—What the Lawrence experiments were to the development of slow sand filters, the results of the investigations at Louisville, Ky., were in respect to rapid or mechanical filters. The late Charles Hermany, Past-President, Am. Soc. C. E., found, as did Mr. Kirkwood at St. Louis, that the muddy Ohio River water was not susceptible to filtration in a practical way through sand beds, as used in Europe and later adopted with some modifications at Lawrence. In 1895 he began investigations, continuing for nearly two years, on various ways of coagulating and filtering muddy Ohio River water, the writer being in immediate charge of the work. Prior to that period mechanical filters had been in use at a few mills, for removing coarse material, but had obtained practically no standing for municipal supplies. They had been studied shortly before by the late E. B. Weston, M. Am. Soc. C. E., at Providence, R. I., and extensive investigations immediately followed those at Louisville, one set being conducted at Cincinnati, Ohio, by the writer and the other at Pittsburgh, Pa., by Mr. Hazen. In those days mechanical filters were contained within wooden tubs or metal cylinders, and it happened that their status became largely enhanced through the development of reinforced concrete for the filter units such as first adapted at Little Falls, N. J., in 1901. Since that date there have been marked developments in mechanical equipment for the efficient control of various filter arrangements.

Filtration Practice.—At present (1926) there are approximately 635 filter plants, having capacities in excess of 1 000 000 gal., serving about 23 750 000

people, and having a total daily rated capacity of 5 000 000 000 gal. There are 47 slow sand filters and 588 mechanical filters, having a rated daily aggregate capacity of 915 000 000 and 4 085 000 000 gal., respectively.

Algæcides.—One aspect of the quality of water has been the tastes and odors associated with the growth and decay of various forms of microscopic life. The late George W. Rafter, M. Am. Soc. C. E., began a painstaking study of this matter, at Rochester, N. Y., and, later, at Boston, Mass., with the aid of the late George C. Whipple, M. Am. Soc. C. E., who for about fifteen years conducted thorough-going investigations into the various factors related to tastes and odors in surface water supplies at Boston and New York. To the engineer, perhaps, it may be of most interest to indicate that by the use of copper sulfate or other algæcides much can be done toward controlling these tastes and odors by killing the organisms producing them.

Aeration.—Stored waters frequently stagnate, with the result that water in the lower parts of deep reservoirs becomes offensive to sight and smell. Aeration will restore the depletion of atmospheric oxygen, lessen the free carbonic acid, and offset the products of decomposition found in the deep parts of large reservoirs. Advantageous use of aeration is to be found in the works of the Catskill project for New York City.

Chlorination.—One of the results of the cholera epidemic at Hamburg was the development of the fact that bleaching powder was an efficient sterilizing agent, although in Europe the idea was to apply it in cases of emergency only. Beginning in 1908, with the Boonton Reservoir supply of Jersey City, N. J., the regular use of chlorine was adopted for the purpose of destroying objectionable bacteria. This practice has become general in America, and it is estimated that there are in service now more than 3 200 chlorinating plants, capable of supplying about 4 000 000 000 gal. of water daily.

Iodizing.—Beginning in 1922 at Rochester, N. Y., there recently has been added to public water supplies intermittent small doses of sodium iodide, for the purpose of supplying the deficiency of iodine. This deficiency in some water supplies is believed to be an important factor in the causation of common goiter.

Methods of Analysis.—The quality and composition of waters in various parts of the United States vary greatly, and the methods of analysis suitable in Western Europe and in New England are quite inadequate for many of the waters of the Southern and Western States. It is of importance to record an unusual amount of good team work in America in improving and unifying methods of analysis. The first edition of "Standard Methods of Water Analysis" appeared in 1905, under the auspices of the American Public Health Association, and, in 1925, its sixth edition appeared under the joint auspices of that Association and the American Water Works Association. It is of interest to note that the standard methods in use in America are to be translated into French and German, under the sponsorship of the Health Section of the League of Nations. It is to be hoped that the co-operative work, carried on in America for thirty years, may assume a wider scope under the guidance of an international committee.

Iron Removal Plants.—Many ground-water supplies contain objectionable quantities of iron and manganese, so that, on exposure to the air, objectionable deposits and discolorations are formed. It is thoroughly practicable to remove these objectionable constituents by aeration and filtration. Chemicals are sometimes used—and in a few instances iron settling basins—for the removal of the deposit, rather than filters.

Removal of Carbonic Acid.—Some water supplies, particularly those derived from deep underground sources, contain large quantities of free carbonic acid, and this is of much significance in connection with the correction of corrosion of metals. It is thoroughly feasible, by aeration and filtration, and, if need be, with the aid of quicklime, to reduce the carbonic acid content to practical limits for safe use. A recent plant for removing such corrosive properties was established for the new deep well supply at Memphis, Tenn.

Water Softening.—There are a few well-known water-softening plants in America. The most comprehensive data have come from Columbus, Ohio, in a paper by John H. Gregory, M. Am. Soc. C. E.* Some supplies of hard water undoubtedly should be softened, but views have not been well crystallized as to where to draw the line separating a hard water justifying softening from one where it is not worth while to pay the cost of such softening. This view, however, for public supplies, does not relate to the opinions held by those in charge of the water supplies of many industrial and power plants. What is said of softening also applies to the removal of carbonic acid and other corrosive properties.

Filter Loading.—Engineers are faced to-day with one problem that very soon must receive more attention than has been accorded to it hitherto. Sources of water supply are being contaminated by industrial wastes to a serious degree in some sections, and the pollution due to the discharge of sewage into such waters after little or no treatment is causing apprehension in well-informed circles. The time is fast approaching when the loads thrown on some water purification plants will be unreasonably heavy, and engineers should consider carefully the advisability of abandoning the old attitude that, except in unusual instances, it is a matter of indifference as to purification of sewage so long as public water supplies derived from a polluted source are purified. In 25 years cities have grown rapidly, and the industries discharging objectionable wastes have grown even more rapidly. Rivers and lakes have about the same volume of water now as they had a century ago. What was good practice in 1900 is no longer such in many cities, because of the changes that have taken place. This view is adopted in the treaty between the United States and Great Britain, ratified in 1909, in which it is stated that boundary waters between the United States and Canada should not be polluted on one side of the river so as to be injurious to those living on the other side. These matters were interpreted to deal with filter loading, after the International Joint Commission, to which these matters were referred, had conducted extensive investigations and inquiries.

* *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), pp. 235 *et seq.*

SEWERAGE AND DRAINAGE OF TOWNS

By Harrison P. Eddy,* M. Am. Soc. C. E.

Historical

Early American Conditions.—At the founding of the United States, the living conditions of the great mass of the people were extremely simple. Plumbing was practically unknown. Few public water supplies and no public sewerage systems existed, and primitive methods for the removal and disposal of household wastes and excretal matters prevailed.

Sewerage and Growth of Cities Coincident.—The development of sewerage systems and methods of sewage disposal has been coincident with the growth of cities. At the time of the first census, in 1790, only four municipalities had populations in excess of 10 000—New York, N. Y., 33 131; Philadelphia, Pa., 28 522; Boston, Mass., 18 320; and Baltimore, Md., 13 503. The remaining population was living under rural or semi-rural conditions. Individual water supplies, in the form of wells, and wastes disposal systems, in the form of privies and cesspools, existed on the same premises, too often in close proximity.

Effect of Introduction of Public Water Supplies.—With the introduction of public water supplies into growing towns, and the gradual increase of houses provided with plumbing, the need for prompt and inoffensive removal of liquid wastes became acute. The pail system, for the collection and removal of excretal wastes, as adopted to a considerable extent in Europe, never became popular in America. Leaching cesspools took care of liquid wastes where soil conditions were suitable, but in many instances offensive wastes were discharged on the ground or into surface gutters. Drains built by individuals or by groups of persons, for the removal of surface waters, were often utilized to carry the liquid wastes to the nearest watercourse.

In Boston it was not until 1823 that the city took over the maintenance of existing drains and the building of new ones. Fecal matters were excluded by law until 1833. Even then, only liquid wastes were allowed, the admission of solid matters being prohibited.

These early drains and sewers were built without regard to the needs of the community as a whole, and did not follow any technical basis of design. The materials of which they were constructed were stone, brick, and wood. Sections were circular, semi-circular, rectangular, and of various odd shapes. In many cases slopes were inadequate, and sometimes actually pitched the wrong way.

Introduction of Water-Carriage System.—Due to the poor design and construction of the early drains there was a marked prejudice against the discharge of excretal matters into them, it being claimed that this would cause offensive odors and be detrimental to health, and it was not until about 1850 that the water-carriage system for liquid wastes, including excretal

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matters, began to come into general use. In Baltimore, the prohibition relating to the direct discharge of such matters into sewers continued until 1911, at which time the new and comprehensive system of sewers and the sewage treatment plant were put into operation.

Yellow Fever and Cholera.—In the early days of the Republic many epidemics of yellow fever and cholera occurred. In New York there were epidemics of the former in 1791, 1795, 1798, 1799, 1805, 1819, and 1822. That of 1798, when the population was only about 80 000, resulted in more than 2 000 deaths.

In Southern cities yellow fever persisted almost until the Twentieth Century. Even before the discovery of the part played by the mosquito in the transmission of the disease, the introduction of sewerage and the drainage of cities resulted in a marked diminution of this malady, due to the reduction of breeding places for mosquitoes.

Cholera, another disease now unknown in America, was prevalent from time to time in the past. Serious epidemics occurred in 1832, 1834, 1849, 1854, 1866, and 1873, thousands of deaths resulting from it in New York alone. A relation between this disease and improper drainage and filthy methods of disposal of household wastes appears to have been recognized to some degree at an early date. This recognition undoubtedly was a stimulus to the establishment of drainage and sewerage systems in the larger towns.

Sewerage Systems

Early Systems.—The first complete systematic sewerage system in the world appears to have been that of Hamburg, Germany, begun in 1843 after the plans of the elder Lindley, a distinguished English engineer. In the United States the first application of engineering skill to the design of sewers was in 1857 when the late Julius W. Adams, Past-President, Am. Soc. C. E., was appointed to prepare plans for the sewerage of Brooklyn, N. Y. For many years thereafter these sewers served as models. In the following year the late E. S. Chesbrough, Past-President, Am. Soc. C. E., submitted his first report on a comprehensive sewerage system for Chicago, Ill. In 1874 the late J. Herbert Shedd, M. Am. Soc. C. E., established the basic principles for the design of a sewerage system at Providence, R. I. Two years later, a committee, consisting of Messrs. E. S. Chesbrough, the late Moses Lane, M. Am. Soc. C. E., and Dr. C. F. Folsom, reported on the sewerage of Boston, Mass., advocating the general plan of main drainage now in effect. In 1880, the late Col. Waring at Memphis, Tenn., and Benezette Williams at Pullman, Ill., constructed the first systems of separate sewers in America. The separate sewerage and drainage systems of New Orleans, La., followed in the early part of the present century, and, in 1911, one of the largest separate systems of sewers in the country went into service at Baltimore. At present (1926) no large municipality is without its sewerage system.

Storm-Water Drains, Separate Sewers, and Combined Sewers.—Following the advocacy of separate sewers by Col. Waring, and with the increasing need for sewage treatment, American practice tended toward the separation

of storm drainage and domestic sewage. Of recent years, however, experience with American municipal administration has shown that it is practically hopeless to secure complete elimination of storm-water from separate sewers and of sewage from storm drains, particularly in the larger cities, and the tendency has been to build combined systems, using intercepting sewers with storm-water overflows when necessary to carry the dry-weather flow a long distance for treatment or disposal.

Pumping.—Although steam-driven pumping machinery had been developed for water-works and for mine drainage prior to the beginning of the Nineteenth Century, there was at that time no comprehensive sewerage system in America, and, consequently, no demand for sewage pumping. Such sewers as were built at an early date discharged by gravity. One of the earliest sewage pumping stations is that of the Boston main drainage works, put into service in 1884. This was followed in the succeeding year by the station in Milwaukee, Wis. In recent years notable pumping plants have been established at Washington, D. C., Baltimore, Chicago, and New Orleans.

The earlier stations were equipped with steam-driven plunger pumps; later, steam-driven centrifugal pumps were used with success, and, more recently, electric motor-driven pumps have predominated. For lifts of only a few feet, screw pumps have been used for many years in connection with flushing tunnels at Milwaukee and Chicago where lake water is pumped to provide additional dilution in sewage-laden streams. At New Orleans very large screw pumps are used for handling drainage or storm run-off. Now that pumping equipment has become more dependable and efficient, and electric current is almost everywhere available, at reasonable cost, engineers do not hesitate, as they once did, to resort to pumping. On both small and relatively large projects, automatically operated equipment is commonly used, thus avoiding the necessity of constant attendance by an operating staff.

Design.—There is little evidence to indicate that prior to about 1850 engineering skill entered into the design of sewers. Following this date empirical formulas for determining their requisite size came into use. Among the best known of these were those of Hawksley (London, 1857); Bürkli-Ziegler (Zürich, 1880); Adams (Brooklyn, 1880); McMath (St. Louis, 1887); and Hering (New York diagrams, 1889). These formulas have given place quite generally to adaptations of the "rational method" for estimating quantities of storm-water run-off. This method, which takes into consideration the fact that the rate of run-off at any point is dependent on the rate of rainfall and the time required for the water to reach the drain, was advocated by the late Emil Kuichling, M. Am. Soc. C. E., in 1889. However, it should be noted that in the modern city the covering of streets and sidewalks with smooth, water-proof pavements and the utilization of a large part of the remaining areas for building purposes, is bringing about a larger proportion of impervious area. These changes cause rates of run-off more nearly equal to rates of precipitation than formerly, resulting in the need for more liberal sewer and drain capacities.

Materials.—The early sewers, built after 1850, were generally vitrified clay or cement mortar pipes for small sizes, and brick for the large sections. In the older part of New York a great deal of wood-stave sewer was used. In 1885, at Washington, David E. McComb, M. Am. Soc. C. E., was the first American engineer to build large sewers of concrete. Since then the use of concrete for large and medium-sized sewers has come into wide use, those at St. Louis, Kansas City, Mo., Detroit, Mich., and Philadelphia being particularly large. In some cases, also, segmental vitrified clay tile has found favor.

Construction Methods.—With increasing activity in the establishment of sewerage systems and in general building construction, and also as a result of advancing wages, there followed improved methods of construction. Excavations for the early sewers were made by hand, stagings being used in deep trenches. This method was then modified by the use of bucket machines, such as those devised by Howard A. Carson, Hon. M. Am. Soc. C. E., during the construction of the Boston main drainage works. Later, on large work, the steam shovel was used; still later, the drag-line excavator; and, on smaller work, the trench excavating machine has come into quite general favor.

For rock excavation, except on small jobs, hand drilling has been replaced by power drills, first of the tripod type and later of the hammer type; and improved explosives have displaced black powder.

Advance has also been made in the methods of handling water encountered in trenching. The old hand-operated diaphragm pump is now driven by a direct-connected gasoline engine, and has been replaced on large work by power-driven pumps of several other types.

Marked progress has been made in the construction of concrete sewers, the hand-turning of concrete having been superseded largely by machine mixing, and collapsible steel forms in many instances having replaced the old wooden ones.

Maintenance.—The early sewers and drains apparently did not receive any attention until stoppages or other troubles occurred, and this is practically true at present in many cities. With the organization of sewer departments in the better managed cities, however, regular maintenance has become more general, and has been facilitated by the development in the past fifty years of improved equipment, such as devices for flushing and for removing obstructions.

Plumbing.—Coincident with the development of public water supplies and the introduction of the water-carriage system for sewage there occurred an evolution in the art of household plumbing. The flush water-closet was invented about 1809, but practically all really sanitary types have been developed within the past forty years.

About 1855 attention began to be given to possible dangers to health from the escape of sewer air or sewer gas. Subsequently, strenuous efforts were made to prevent even the most minute escape of sewer air into houses, and there ensued an era of complicated plumbing layouts fostered by elaborate plumbing codes.

The first important plumbing legislation in Massachusetts was in 1882 when the City of Boston was authorized by the Legislature, to regulate plumbing by ordinance. In 1924, the late George C. Whipple, M. Am. Soc. C. E., as chairman of a sub-committee, reported to the Department of Commerce on minimum requirements for plumbing. This report described the situation relative to plumbing codes as chaotic, and disclosed an utter lack of uniformity in legal procedure governing such matters. The findings of this committee marked an important step toward increased standardization and simplification of plumbing practice.

Sewage Disposal

Stream Pollution.—In the early days filthy privy vaults and overflowing cesspools in populous districts led to the construction of common sewers, which discharged generally into the nearest stream or other body of water. By this practice the nuisance at the individual premises was transferred to the point of discharge of the sewer, resulting in an extension of the offensive conditions to the stream or along the water-front. The construction of intercepting sewers with a single outlet, in turn, served to concentrate the polluting material, and with increasing volumes of sewage the intensity of pollution in many instances became intolerable.

In 1872 the State Board of Health of Massachusetts was authorized to investigate the problem of stream pollution, and in the following year William R. Nichols, Professor of Chemistry of the Massachusetts Institute of Technology, submitted to the Board a comprehensive report on sewerage, sewage disposal, and stream pollution. In the same year the late Phinehas Ball was engaged in a study of the Worcester sewage problem, and in 1875 the late J. P. Kirkwood, Past-President, Am. Soc. C. E., made an investigation and report on stream-pollution conditions in Massachusetts. Since that time numerous investigations and reports have been made by other investigators and also by State and Federal departments.

Legislation and Administrative Control.—Nearly all the States have passed legislative measures intended to restrict and control stream pollution. Some of these laws and regulations have been so drastic as to be impossible of strict enforcement. Recently, however, a tendency toward a more rational regulation of the use of streams has developed, as illustrated by the formation of the Sanitary Water Board of Pennsylvania and its classification of streams in three groups, with restrictions for the disposal of sewage and industrial wastes applicable to the needs of each group.

In general, the control of stream pollution has been vested in the respective State departments or boards of health. In the case of interstate streams, only the Supreme Court of the United States has jurisdiction. There have been, however, a few agreements between the departments of health of several States. The first of such agreements was that in 1922 between the Health Departments of Pennsylvania and New Jersey, with respect to the Delaware River. Since then the Health Departments of Pennsylvania, Ohio, West Virginia, Kentucky, and New York have executed general agreements relating

to the streams passing through these States. In this manner it may be possible, by exercising their respective local authority, for the several States jointly to accomplish control of pollution of interstate waters without recourse to Federal authorities.

Scientific Research.—In 1880 the discovery of the bacillus of typhoid fever, by Eberth in Germany, marked the beginning of a new era in sanitation. Previously, the relation of pollution to disease had been but faintly understood, such sanitary works as were built being due to a desire for cleanliness and a sentiment of common decency. Although such motives remain in force to-day, and in fact in many instances govern, the most vital consideration now is "prevention of disease."

Fifty years ago the science of bacteriology was in its infancy, and its application to matters of stream pollution and sewage disposal had not been grasped. In 1877 Shloesing and Muntz, in France, and, in 1882, Robert Warington, in England, proved conclusively that the oxidation of ammonia and organic matter was effected by the agency of living organisms, and Warington proceeded to devise practical methods whereby living organisms could be utilized for the nitrification of the organic matters in sewage.

In 1887 the Lawrence Experiment Station of the Massachusetts State Board of Health was established, under the direction of the late Hiram F. Mills, Hon. M. Am. Soc. C. E. Through its studies the fundamental biological conditions underlying the oxidation processes of sewage treatment became established. These researches have been continued to the present day (1926), constituting an unparalleled contribution, from a single source, to the art of sewage treatment.

Dilution.—Although sewage disposal by dilution was the earliest method adopted by municipalities, it was not until about 1887 that investigation of the factors entering into it was made by the late Rudolph Hering, M. Am. Soc. C. E., in connection with the problem of sewage disposal at Chicago. His studies resulted in the construction of the Main Drainage Canal of the Sanitary District of Chicago, completed in 1900. In Massachusetts studies undertaken at about the same time established the limiting dilution ratios which served as a basis of general practice for many years. More recently much information bearing on the disposal of sewage by dilution has been accumulated. Particularly noteworthy have been the studies made by the United States Public Health Service in connection with the Ohio and Illinois Rivers.

Although seacoast municipalities have seldom adopted methods other than that of dilution, it has often proved necessary, even with them, to take precautions with respect to the location of outlets, so that pollution of shores and bathing beaches should be prevented. In the case of the Boston main drainage works, for instance, special attention was paid to the location of outlets, the sewage being temporarily stored, and released only at ebb tide. In the case of certain seacoast towns where it is desirable to protect shellfish areas from contamination, treatment of the sewage has been adopted, with the addition of disinfection.

Broad Irrigation.—The treatment of sewage to render it inoffensive was undertaken even before the principles involved began to be understood. The

earliest method was that of broad irrigation, or the application of the sewage to relatively level areas of land, where it could become absorbed and purified by the soil. To operate satisfactorily, large areas of porous and easily drained soil were required, and, to avoid annoyance from flies and odors, it was necessary that the "sewage farms", as such tracts were called, be well isolated. Usually an effort was made to utilize the manurial value of the sewage by growing crops on the irrigated fields. Among the first attempts of this sort in the United States may be mentioned the farm established at Lenox, Mass., in 1876, where both subsurface irrigation and broad irrigation were used; the broad irrigation areas at the State Insane Asylum in Worcester, in 1876; at Amherst, Mass., and at Pullman, Wash., in 1881; and at South Framingham, Mass., in 1889. Due to the large tracts of land required, however, and also for esthetic reasons, this method of treatment never was adopted widely in the United States, and, with a few minor exceptions, is no longer in use.

Intermittent Filtration.—As a result of the Lawrence Experiment Station studies there was developed in New England a modified form of broad irrigation, known as "intermittent filtration," by which crude, or settled, sewage is applied evenly to the surface of prepared areas of sand or other fine material a few feet in depth, underdrained by lines of tile with open joints. During its passage through the bed the sewage is purified as a result of the removal, and changing, of the organic matter into more stable substances, by physical and biological agencies in conjunction with atmospheric oxygen present in the interstices of the sand. The process derives its name in part from the necessity of intermittent application of the sewage in order that the air required for the oxidation of the organic matters may enter the voids of the sand. In New England, where soil conditions are favorable, this method was promptly adopted by many towns. The area required is much less than for broad irrigation, but greater than for later types of filters. Owing to the large area required, and the cost of operation, this method is adopted now only for small towns where conditions are favorable.

Chemical Precipitation.—Another early method of sewage treatment is that of "chemical precipitation", involving the addition of lime, lime and sulfate of iron, or other coagulants, to form an inorganic floc, which absorbs and on settling carries down with it particles of suspended solids, leaving a relatively clear liquid. The sludge produced is large in volume and somewhat offensive in character.

The earliest plants of this type were those at Coney Island, New York, put in operation about 1887; at East Orange, N. J., in 1888 or 1889, and at Worcester, in 1890. Later, other plants of this type were built at various places, the most recent large one being that at Providence, built in 1902. Due to the incomplete purification secured, the high cost of operation, and the large volume of sludge produced, chemical precipitation as a means for treating sewage, has gone out of favor, although it is still recognized as of value in the treatment of some industrial wastes. Recently, the City of Worcester abandoned this method of treatment and, in 1925, put into operation a large Imhoff tank—trickling filter plant.

Septic Tanks.—The sedimentation of sewage in tanks, without the use of chemicals, has been practiced for many years. The combination of the sedimentation of solids and their digestion by bacterial action has been utilized in the septic tank, the Travis tank, and the Imhoff tank.

The septic tank is a sedimentation tank, designed and operated so as to foster the decomposition of the settled solids in the absence of oxygen. It was originally believed that nearly complete liquefaction of the suspended solids could be obtained in this manner. Experience has shown, however, that the proportion thus liquefied is much less than was formerly thought. Such tanks, unless covered, frequently are offensive smelling, the treated sewage is black and foul, and difficulties in operation result from the accumulation of the solids in the form of a floating mass of scum on the surface of the sewage.

Similar tanks were first used in America at the State Insane Asylum at Worcester in 1876, and later were adopted at other places. An installation at Saratoga Springs, N. Y., in 1903, led to a lawsuit brought by the owners of certain septic tank patents, which resulted in judgment for the plaintiffs. From the beginning of septic tank patent litigation the consensus of opinion among American sanitary engineers has been that the patents covered principles and practices which had long been known and publicly used. Consequently, under these patents, which long since have expired, the payments were few and small.

Imhoff Tanks.—The favor in which septic tanks were once held later turned largely to the two-story tank in which the processes taking place in the septic tank are separated, sedimentation occurring in an upper compartment, and digestion of the solids going on in a chamber below, separated from the one above by a sloping partition containing narrow slots through which the solids pass into the lower compartment.

This tank was developed for the Emscher District, in Germany, and was introduced into this country largely through the efforts of Mr. Hering.

The earliest plant in America was that at Madison-Chatham, N. J., in 1911, and the first large tanks were constructed at Atlanta, Ga., in 1912. Since then there have been many important installations, among them those at Rochester, N. Y., Fitchburg, Mass., Cleveland, Ohio, and Philadelphia. This tank is a material improvement on the older septic process in that the treated sewage is less offensive and the solids may be digested to such a degree that the resulting sludge is practically free from odor and can readily be dewatered on porous beds. In some places, however, difficulties have been experienced due to unfavorable sludge digestion, which has resulted in foaming or in excessive scum formation. Recent work, at the New Jersey Sewage Experiment Station and at other places, indicates that by controlling the reaction (pH) of the sludge and scum, proper digestion can be secured.

Separate Sludge Digestion.—It has been held by some engineers that the processes carried out in the two-story tanks can be accomplished better in two separate tanks, the solids being settled out in one and drawn or pumped from it into the digestion tank. In some cases the sedimentation tanks have been

equipped with mechanical devices for the continuous removal of the settled solids.

The earliest attempt at separate sludge digestion on a large scale in America was at Baltimore in 1912, where open tanks were introduced for the storage during digestion of solids removed from preliminary settling tanks. On the whole, the Baltimore experience, although not yet fully satisfactory, has demonstrated that sludge may be digested in separate tanks and that this process may prove of advantage in some cases, if conditions favorable to the requisite biological action can be provided and maintained.

Separate sludge digestion generally has resulted in acid fermentation accompanied by offensive odors and the production of a foul-smelling sludge. Research at the New Jersey Sewage Experiment Station and at Harvard University has shown the importance of controlling the reaction of the sludge and the advantage which can be secured by maintaining a favorable temperature. At Plainfield, N. J., experiments relating to this subject have been carried out on a large scale, and at Boonton, N. J., a plant has been constructed in which provision has been made for collecting and burning the gases from sludge digestion and for utilizing the heat therefrom for heating the glass-housed sludge beds and the contents of the separate sludge digestion tanks.

Contact Beds.—Because of the large areas required, and the unavailability of suitable soil for broad irrigation or for intermittent filtration, coarse-grained sewage filters, in the form of contact beds, originated in England in the early Nineties and were adopted somewhat later in America. Contact beds consist essentially of tanks, filled with broken stone, coke, or other coarse medium, which are alternately filled with settled sewage and emptied, periods of several hours being allowed for standing full and remaining empty.

Oxidation and nitrification are brought about by the film of micro-organisms growing on the surface of the medium, the supply of oxygen being obtained from the air entering the voids of the material during the draining period. Comparatively few contact beds have been built in America, and several of those have now been replaced by other forms of treatment. This was due to the almost coincident development of trickling filters, which are superior to contact beds in some respects, chief among them being the much larger volume of sewage which can be treated on a unit area of trickling filter.

Trickling Filters.—Trickling filters, also adopted in England on a large scale prior to their use in America, differ from contact beds in that the sewage is distributed more or less uniformly and continuously over the surface of beds of coarse material. A trickling filter is primarily a bed of broken stone several feet deep, laid on a concrete floor covered with a system of underdrains. Settled sewage is applied intermittently, at short intervals, to the surface of the filter in the form of fine drops, by using symmetrically spaced spray nozzles or other devices, comparatively few mechanical distributors, such as are commonly used in England, having been built in America. As the sewage is sprayed through the air and passes through the porous bed, atmospheric oxygen is absorbed and is utilized in the biological oxidation and

mineralization of the organic matter. The effluent is usually settled in order to remove humus-like solids which, from time to time, escape from the filter and thus prevent it from becoming clogged.

The first municipal filter of this type to be put in service was that at Reading, Pa., in 1908. At present, trickling filters preceded by Imhoff tanks, probably constitute the type of treatment most generally in use in America. The largest installation of such filters is that at Baltimore, where about 30 acres are in use.

One unsatisfactory feature of the trickling filter is the occurrence at times of large numbers of the small gray fly, *psychoda*. Control of these flies has been accomplished by flooding the filters for brief periods at regular intervals, during the breeding season, although this has interfered somewhat with the efficiency of the filters. At Schenectady, N. Y., during the last two years, considerable success has attended an effort to control this fly by treating the settled sewage, just prior to its application to the filters, with either bleaching powder or liquid chlorine. The control of odors around the Imhoff tanks and the trickling filters, by the treatment of the sewage with liquid chlorine, is also being investigated, and gives promise of beneficial results under some conditions.

Activated Sludge.—During the past ten years the activated sludge process has been adopted in a number of places in the United States, Canada, and abroad. The process is the outcome of experiments on the aeration of sewage, by numerous American workers, particularly the investigations at the Lawrence Experiment Station in 1912, which led to the work, in England, of Fowler and Mumford in 1913 and of Arden and Lockett in 1914. This treatment consists essentially of aerating sewage with a mixture of aerated or activated sludge and subsequently allowing the mixture to settle. The development of the process as a practical means of treating sewage has resulted largely from investigations made at Milwaukee, Chicago, Indianapolis, Ind., and several other places.

Beginning in 1914, the City of Milwaukee conducted what is undoubtedly the most extensive large-scale experimental investigation ever carried out in the field of sewage treatment. Based on the results of these experiments, a plant, the largest in the world at present (1926), has been constructed at Milwaukee, which was put into operation in 1925. At present there are comparatively few activated sludge plants in America, that at San Marcos, Tex., built in 1916, having been the first. Large plants have since been constructed at Houston, Tex., Indianapolis, and Chicago. In the latter city a plant for the treatment of the sewage from a population of 800 000 is now under construction.

Racks, Grit Chambers, and Screens.—In order to protect pumps and other mechanical equipment from clogging and injury, and to simplify and improve the efficiency of various processes of treatment, it has become general practice to provide preliminary treatment for the removal of trash, mineral matter, and the coarser suspended organic substances. In a few cases provision has also been made for the removal of floating oil and grease.

Cage racks for the interception of trash were constructed at the main drainage pumping station in Boston and first operated in 1884. Grit chambers for the removal of the heavier mineral solids were built at Worcester in 1904. Stationary bar screens, manually and mechanically raked, have been in use for a long time, but only during the past 15 or 20 years has there been a marked development in fine, moving screens cleaned by brushing or flushing. The first of these in America was the Weand screen built at Reading, in 1908. This was followed by the Riensch-Wurl screen at Rochester in 1916 and later by others, such as the Dorrco and Link Belt screens, at various places. During the last few years such screens have been used for treating sewage preparatory to more complete treatment by other processes, and for removing the larger and more easily visible floating and suspended matter where more complete treatment is not necessary.

Sludge Disposal.—One of the troublesome features of sewage treatment has been the disposal of the solid particles removed by screening or by sedimentation. In general, screenings have been used for filling, or have been plowed under in an effort to utilize their fertilizing properties. To a limited extent, also, screenings have been incinerated. Sludge from septic and single-story settling tanks is generally offensive and difficult to dry satisfactorily on beds of sand or similar material. Properly functioning two-story tanks produce sludge which is easily dried and inoffensive. As rains and freezing weather interfere with drying in the open air, beds have been built with covers, like greenhouse structures, notably at Marion and Alliance, Ohio. At Houston and Indianapolis activated sludge has been lagooned.

During the early years of the treatment of sewage, attempts were made to utilize its manurial value by growing crops on the areas used for broad irrigation. Later, there was considerable effort to utilize the fertilizing ingredients in the sludge resulting from chemical precipitation. In some cases such sludge was dewatered by the common filter press, notable plants being at Worcester and Providence.

Recently, air-dried digested sludge has been sold to farmers at Baltimore, Rochester, and Schenectady. The value of such material is small, because of its high water content and the low proportion of fertilizing ingredients contained in it. In some localities it appears to have been profitable for farmers to haul and utilize such sludge, even if a nominal price had to be paid for it.

In the case of activated sludge, the proportion of fertilizing ingredients, particularly nitrogenous compounds, is materially greater than in sludges from other processes, and, therefore, the opportunity for its profitable utilization as a fertilizer has proved somewhat attractive.

By far the greatest progress in this direction has been made at Milwaukee, where a large plant has been built for dewatering and drying activated sludge, thus converting it into a marketable fertilizer which is being sold under the name of "Milorganite." At Chicago, also, for several years, activated sludge has been dewatered, dried, and converted into fertilizer.

Disinfection.—Where treated sewage is tributary to public water supplies or where effluents are discharged into tidal waters used for oyster culture, it

has become customary to provide for disinfection with chlorine. The first plant for this purpose was at Brewster, N. Y., on the Croton water-shed, where an electrolytic plant for the production of chlorine from salt solution was built in 1892. In 1909 the efficiency of hypochlorite of lime for the disinfection of sewage, was demonstrated, and, at present, Providence is using this agent to disinfect the effluent from its chemical precipitation plant. Liquid chlorine, however, is used more generally. Chlorination plants have been installed at Cleveland and at a number of other places where raw or treated sewage is discharged into waters used for bathing.

Industrial Wastes.—England, due to her smaller streams, was compelled to devote attention to the treatment of industrial wastes, as well as of sewage, long before it became urgent to do so in America. Rafter and Baker, in their book, "Sewage Disposal in the United States," published in 1894, state as follows:

"Thus far in this country manufacturers have not, except in a few instances, been compelled to purify their own polluted wastes, and inasmuch as few have undertaken the purification, of their own volition, very little experience under American conditions has thus far been gained."

Even to-day the foregoing statement holds true in considerable degree, although that part dealing with the lack of experience under American conditions should be modified. In New England and elsewhere numerous plants have been built for the treatment of industrial wastes. Furthermore, manufacturers are less wasteful than in previous days, and the quantities of polluting matters discharged in the waste waters are in some cases somewhat less. In general, the processes used in treating industrial wastes include sedimentation, chemical precipitation, intermittent filtration, oxidation in trickling filters, or, in short, processes or combinations of processes which have been found applicable to the treatment of sewage.

Increase in Sewage Treatment.—Fifty years ago not one of the sixty-seven cities having a population of 100 000 or more by the 1920 Census, used any method for the treatment of its sewage. By 1900 only two of these cities—Worcester and Reading—had built treatment plants, their aggregate population then being approximately 197 000. At present (1926) about twenty of these cities, with a total population of approximately 6 500 000 by the 1920 Census, have treatment plants. Of these plants those receiving the sewage from a population of about 3 700 000 consist of tanks only, or of tanks preceded by screens; the remaining plants, receiving sewage from about 2 800 000, provide for tank treatment combined with some oxidation process, such as activated sludge or trickling filters.

The change in the art of sewage treatment chiefly by biological processes is illustrated by a comparison of the space required by different types of plant for the treatment of a unit volume of sewage—for example, that from a city of 100 000 population. The early method of intermittent filtration, including settling tanks, sludge beds, and sand filters, would need a total of not less than 130 acres and 23 000 000 cu. ft.; the more modern Imhoff tank—

trickling filter plant, with sludge beds and appurtenances—would require about 10 acres and 3 000 000 cu. ft.; and a plant for the most recent activated sludge process would require about 2 acres and 1 000 000 cu. ft.

Operation of Treatment Plants for Sewage and Industrial Wastes.—Although much progress has been made in the art of sewage treatment, and many sewage treatment plants have been built, there remains considerable lack of realization, on the part of the public and of municipal officials, of the necessity for continuous, careful, and intelligent operation of such plants. Prior to 1900 only about two sewage treatment plants in the United States were under the control of resident technical operators, these plants being at Brockton, Mass., and Worcester. By 1910 several other treatment plants were under technical supervision, notably at Columbus, Ohio, Plainfield, and Reading. Since that time many new plants have been constructed, of which the larger have been provided almost invariably with technical operators. The smaller plants, however, in far too many instances, are operated without proper supervision.

Technical supervision of plants for the treatment of industrial wastes is even less common, largely because such plants are a source of outgo rather than of income, and, consequently, do not enlist the interest of the proprietors. On the other hand, some progressive manufacturers have provided for the expert technical supervision of intelligent non-technical operators, with excellent results.

Summary

Reviewing the history of the last 150 years, it is seen that the advance in the development and application of the art of sewerage and drainage of towns, although slow at first, has attained such an acceleration that there is a wide gap between the conditions as of the beginning and the ending of the sesqui-centennial period.

In the early years the privy, the cesspool, and the drain to the nearest watercourse prevailed. Cities were without water supplies and without sewers. The well and the cesspool or privy existed side by side. Visitations of cholera and yellow fever occurred from time to time, and typhoid and malarial fevers were prevalent. The relation to disease, of parasites conveyed by food and drink, or by insect hosts, was unknown. The general death rate was abnormally high, viewed in the light of present conditions, and little attention was given to the sanitation of such towns and cities as existed.

During the latter part of this period, discoveries in the science of bacteriology, relative to the causation of disease and the various methods of its transmittal, have resulted in rapid strides in the sanitation of the environment. The prevention of pollution of water supplies, their purification in case of contamination, and the proper removal and disposition of the waste products of human activities, have made remarkable progress. Resulting in large measure from the application of scientific knowledge, coupled with engineering skill in the control of man's environment, cholera and yellow fever have dis-

appeared, typhoid fever and other water-borne diseases have almost been eradicated, and the general death rate has been diminished materially.

Progress in the field of sewage disposal has been more spectacular than in those of sewerage and drainage, using these terms in a restricted sense. Even fifty years ago, the principles of sewage purification, either solely by natural means or by artificial processes utilizing natural agencies, were not known. To-day, many of these principles are known, and processes are available by which municipal sewage can be purified practically to any degree desired.

The fundamental reason for progress has been the ever-broadening development of science and its application to the art of sanitary engineering. Such application of knowledge has been greatly stimulated by the ever-increasing growth in population and the aggregation of this population into communities of restricted area.

The Future

If it were possible to project a curve of the accumulation of knowledge, it would be less hazardous to prophesy future developments.

In the field of sewerage and drainage there does not seem to be any reason to anticipate radical departure from the principles now established. Data useful as a basis of design, however, will become more complete, and the application of engineering skill will result in better design and construction as time goes on.

Household plumbing will be somewhat simplified, but there will be a still further increase in plumbing conveniences and a greater volume of household sewage. This increase, however, will be offset in some degree by general metering of water supplies and by greater economy in the use of water in the industries.

With continued increase in density of population there will be greater demand for the proper regulation of stream pollution. It is probable, however, that the comparatively recent tendency in some places to attempt a complete cleaning up of streams, will give way to a policy of reasonable use, subject, however, to far better control than is exercised in most localities at present.

With an increasing number of workers in the field, it is certain that better methods of sewage treatment will be devised and developed. The progress of the last fifty years has been confined primarily to the utilization of biological forces, the means of accomplishing which will be greatly improved in the future. The most promising opportunity for the development of new and better methods now appears to lie in the field of physical chemistry, research in which may show a way by which organic colloids can be removed from sewage without the aid of biological agencies. Such a development may lead to simpler methods, less susceptible to temporary derangement. Although it seems improbable that a net income will be derived from the recovery of the fertilizing ingredients of sewage, it is probable that sewage sludge will be utilized as fertilizer in an increasing degree.

The serious handicaps under which most sewage plants are operated to-day, due to lack of adequate funds and proper management, will be overcome in the course of time. The most promising means of accomplishing this is State legislation, of which the Ohio Act of April 5, 1923, is a forerunner. This provides for sewer rentals to cover the cost of operation and maintenance of sewerage works.

The work accomplished during the past 150 years in the field of sewerage and drainage of towns has been the result of the effort of relatively few individuals. Looking backward, the achievements appear to have been great, notwithstanding the difficulties resulting from lack of knowledge. The future increase in population and the raising of the standards of sanitation will afford large opportunities. Looking forward, it appears that the number of workers in the field is destined to increase, and that the accumulation of knowledge in science and in the arts may well enable the engineers of the future to accomplish results even more remarkable and beneficial than those of the past.

STREET CLEANING AND THE COLLECTION AND DISPOSAL OF REFUSE

By Samuel A. Greeley,* M. Am. Soc. C. E.

General Statement

The cleansing of cities involves the removal and disposal of the solid wastes resulting from the activities of the inhabitants. These wastes collect in various ways on the surfaces of streets, yards, vacant areas, and roofs. Such solid wastes, commonly termed refuse, include all the products of community life except sewage. Refuse materials comprise garbage, ashes, rubbish, street dirt, manure, trade refuse, dust, etc. By manual labor and by the use of wagons, trucks, and other special equipment, refuse materials are collected and disposed of.

In the absence of regular and frequent removal, such materials accumulate principally on streets and in alleys, and to a less extent on vacant areas and in yards. There has been, and still is, a marked tendency on the part of individuals to throw their refuse into the streets. In the early days, the hog was the common scavenger, and roamed about and partly cleaned the city. The following was written in the early Fifties:†

"Our streets and alleys should be cleaned everywhere, especially in and around the neighborhood of the square. The brick bats, trash, old hats, old boots and shoes, rags, bones, manure and many other things which grace our streets, should be hauled off, and hog holes filled up."

The need for final disposal was not yet clearly recognized. People in Quincy, Ill., considered geese better scavengers than hogs, and so let them roam the town.

The rapid growth of cities has made for a constantly changing problem in connection with city cleaning, and it has been difficult from the average point of view to keep pace with the necessary demands.

Gradually, people began to understand that the accumulation of refuse is unhealthy. Dr. Frank M. Wells, Health Officer of Cleveland, Ohio, in 1876, wrote:

"It is a sanitary necessity that cities should be kept clean and their refuse, which is organic matter in a state of decomposition, should be promptly removed. This can be accomplished with greater facility when streets are paved, and therefore the paving of streets demands the attention of sanitary bodies. * * * The importance of a constant and thorough cleaning of the streets can not be over-estimated. * * * Of more importance, perhaps, than the condition of the streets, is that of the yards connected with the dwellings, since they are in such close relation with the house that the effects of filth deposited in them is more readily felt."

Refuse left without removal promotes the breeding and growth of vermin, mosquitoes, flies, and rats, all of which tend to spread diseases among human

* (Pearse, Greeley & Hansen), Chicago, Ill.

† In *The Journal*, Springfield, Ill.

beings. During certain seasons, the refuse dries, and is pulverized to dust, which fills the atmosphere, to the annoyance of the people and often to the great damage of merchandise. Such influences are directly proportional to the quantity of refuse and the density of population. The quantity of refuse per unit or area in a community increases with the density of the population, and the effect of such accumulations on public health and comfort is greater in crowded, heavily populated districts than in open rural communities. Thus, the development of sanitary engineering in the field of city cleaning during the past 150 years follows closely the development of cities and the sharp increase in urban population.

Growth of Cities

The first considerable growth of cities in the United States began during the twenty years preceding the Civil War. The winning of the Mississippi Valley, the building of railroads, sundry improvements in farm machinery, and successful invention earned an era of prosperity and stimulated industry. Cities grew. Following the Civil War and the development of the West, factories were built and people flocked to the cities. The increasing density of population in several large cities is shown by Table 3, the data for which are taken from the U. S. Census.

TABLE 3.—Density of Population in a Few American Cities.

City	Population per Acre.	
	1900	1924
New York, N. Y.	18.6	31.5
Philadelphia, Pa.	15.5	24.2
Boston, Mass.	21.4±	28.3
Chicago, Ill.	13.9±	24.1
St. Louis, Mo.	14.7	21.0
New Orleans, La.	2.3	3.6
Average	14.4	22.1

This concentration of population forced the adoption of special measures for city cleaning. In some notes on street cleaning in the Common Council Manual of New York City (1862), it is stated:

"In May, 1701, the assistant aldermen of the different wards were instructed to call on the several inhabitants in their respective wards and ascertain what they are willing to pay towards cleaning the streets and carrying away the dirt; but this method not meeting with a proper response on the part of the inhabitants, an order was made in the same month that the inhabitants of the city, on every Friday, sweep the dirt in heaps before their respective premises * * * for removal by the city cartmen."

In 1887, the Ladies Health Protective Association addressed a memorial to the Mayor of New York, stating that "Modern science has fully exposed the imminent dangers that beset health and life in a malodorous and dust covered

town" and, after reviewing methods of city cleaning, urged that "the street cleaning system of New York be wholly reorganized, or that a far different spirit of energy and efficiency should be infused into it."

Thus, if the need for city cleaning has grown with the cities, the effectiveness of city cleaning has developed more or less consistently with progress in municipal government and the enlightenment of public opinion. The contrast between the cleanliness of New York City under the administration of the late Colonel Waring and during the next four years under Tammany as described by a sanitary engineer, William F. Morse, of New York, may be cited as follows:

"Beginning in 1898, with the inheritance of a well-organized and thoroughly equipped service, with labor and money-saving devices and apparatus in running order, with plans and purposes well-defined for carrying on a practical and successful line of work in an honest and economical way; now at the end of four years, as the result of incompetent management and complete surrender to the machine politicians, the department is in a position of absolute contempt."

Progress has been secured in growing cities, despite periods of stagnation caused by incompetent management. The sharp reduction of the general death rate from around 25 per 1 000 about 50 years ago to less than 15 at present (1926), has been caused by improved city cleaning as well as by advances in water supply, sewerage, and public health administration.

Elements of Progress

Broadly speaking, progress in city cleaning depends on the degree of excellence of municipal government, and on the methods and equipment developed by sanitary engineers for use by city officials. It may be said, then, that two of the major elements of progress in this field are (*a*) the intelligence and character of public opinion, as it determines the character of the municipal government; and (*b*) the zeal and skill of outstanding individuals leading the way beyond their contemporaries. The latter provides the facilities for improved methods of city cleaning and the former marks the extent of efficiency in operation and results.

The excellence of municipal government is, of course, largely an outgrowth of public opinion. The more intelligent, experienced, and outspoken the public opinion, the better, in the long run, will be the municipal government. This, in turn, stimulates the science and practice of city cleaning. Outstanding events which picture clearly to the average person important factors of public health mark forward steps. The picturesque descriptions of the work of Pasteur among cattle and silk worms are typical. Perhaps in America the great losses from typhoid fever spread by flies from privies during the Spanish-American War stirred public opinion toward better standards of municipal cleanliness more than any other single event. The conquest of yellow fever by Gorgas resulted from improved cleaning of cities as well as through drainage, and demonstrated to the public the gain in public health through public hygiene. During the quarter century following the Spanish-

American War, progress in city cleaning was more marked by far than at any other time.

Among the names of those active in city cleaning work within memory are Wells, Waring, Landreth, Parsons, Goodnough, Whinery, Morse and Hering. Such men conducted scientific investigations, studied experience here and abroad, and applied their knowledge to actual operation. One of the first comprehensive investigations of the subject was begun in 1887 by the Garbage Committee of the American Public Health Association, of which the late Rudolph Hering, M. Am. Soc. C. E., was Chairman. One of the best early efforts in this direction was by Dr. Frank M. Wells, Health Officer of Cleveland, in 1876. In a notable report to the Board of Police Commissioners entitled "Filth and Its Relation to Disease", Dr. Wells states the fundamental aspects of city cleaning. After limiting his subject matter to sewers and scavenging, he disposes of the then conflicting theories of disease—one holding that diseases spring spontaneously from filth, and the other that filth furnishes a favorable atmosphere for the multiplication of the organisms of disease—by the following statement:

"Practically these two theories are identical since there is but little difference, whether filth directly causes disease, or is an essential factor in the development of the organisms which produce disease.

"That unsanitary conditions bear a certain relation to a particular class of infections, is now the opinion of the majority of the medical profession."

Early Practice

The beginnings of city cleaning were undoubtedly in street cleaning. Residents deposited their refuse in piles in the street whence it was removed, by a public scavenger, to a public dump beyond occupied districts. Roaming hogs disposed of the garbage they could eat. Thus, hog feeding of garbage and land fill had their beginnings. Practice in Cleveland in 1876 is thus described by Dr. Wells:

"The refuse, which is usually deposited in the yards, consists of ashes, swill, and garbage, which, under the present ordinances governing these articles, form, I believe, a very prolific source of filth. These ordinances provide that no ashes shall be mingled with the swill and garbage (the very means which would render them inoffensive), and that they shall be removed by the Contractor not less than two times a week during the months of May, June, July, August, September, and October, and as many times oftener as may be necessary to prevent the swill deposited from becoming offensive; and not less than once a week during the rest of the year, upon notice from the householder being sent to him. By another section, every householder is obliged to provide water-tight receptacles for the swill and garbage which may accumulate during 48 hours."

Practice, prior to 1887, and its trend for the future, is outlined in the Memorial of the Ladies Health Protective Association as follows:

"Our third point is, that neither ashes nor garbage should ever be placed on the pavement, but that, like the mail carriers, the collectors of house refuse should have a whistle to warn of their coming, and that when they ring, the cans should at once be taken out of the house, emptied into the cart, and returned at once into the house. We think that refuse carts should have high sides and a sloping cover with a hopper in the center, and that every house-

holder should be required to own a galvanized iron receptacle fitted with a cover and with a sliding bottom, so that when placed over the hopper and the slide pulled out, the contents will be received into the closed cart, and the now intolerable blowing of ashes and dust about the streets from the carts be thus avoided. We are agreed that, to get rid of the house ashes and garbage, and of the street sweepings, crematories should be built, similar to the so-called 'Destructors' and 'Carbonizers' in use in England, and that the house ashes and garbage should be burned in the Destructors and the street sweepings in the Carbonizers, and the residuum used for making mortar in one case and fertilizers in the other, as is now so successfully done in Leeds and other English cities. The system at least pays for itself, and if adopted here, the wasteful, pernicious, and disgusting practice of dumping city refuse either inside or outside of the magnificent bay of New York, might be wholly discontinued."

Dr. Henry I. Bowditch, of Boston, Mass., gathered statistics for the Centennial Exposition in 1876 from about seventy cities in the United States, and summarized them as follows: In about twenty-five of the seventy cities, the house offal is moved in carts. The remaining forty-five places report a variety of methods, such as: (1) Thrown into cesspools; (2) fed to hogs; (3) thrown into river; (4) dumped in valley; and (5) used as fertilizer. Thus, less than one-half the cities reporting appeared to have a regular collection service.

The collection and disposal began with dumping refuse on the streets, whence it was gradually removed by hogs, scavengers, and the disintegration of traffic. Then came removal and dumping by the municipality, the householders being required to provide cans. Special methods of disposal began in the decade, 1880 to 1890. In 1873 a report to the American Public Health Association by Dr. C. A. Leas, of Baltimore, Md., states as follows:

"* * * Again, in other localities, the scavenging service is wholly or partly intrusted to Contractors, who mainly regard their own pecuniary interests rather than the public health. The municipal authorities of some towns and cities employ scavenging forces, but without fixed regulations. * * * In some cities the collection of kitchen offal and house ashes is made separately by two sets of carts, with a view to the utilization of the ashes for the filling of sunken lots, and the vegetable and animal offal for the feeding of swine. * * * Whilst in others it is collected together. * * *."

The report then describes a complicated system of composting the garbage with sifted ashes, a tank treatment of night-soil, with a decanting of the liquids to the compost pits, and the final preparation of a fertilizing material. This process appears to have been tried at Baltimore at about this time, but evidently was not continued.

General Development Since 1876

The condition of the streets and the collection of refuse are closely related to the habits and actions of the residents and housekeepers. Their co-operation is important. The general development of city cleaning has depended a great deal on the increased understanding of the public. Much educational work has been accomplished by street-cleaning departments and those charged with the collection and disposal of refuse. The growing familiarity of foreign sections of the population of American cities with city cleaning rules and regulations and the efforts of the cleansing departments have facilitated and

improved the work. Great numbers of kitchen cards, based on ordinances regulating the house treatment of refuse, have been distributed. The separation of refuse materials at the house, where ordered, is much better secured. Although not so readily landmarked, better co-operation by the public has contributed largely to the progress of city cleaning in the last half century.

Street cleaning and collection equipment has been developed with the advance in manufacturing and the development of motor vehicles. Perhaps even more important is the better understanding gained by experience in operation, of the fundamental principles involved. Frequent time studies of the work of collection units have been made, and careful consideration of the relations between the size of the wagon, its loading height, the location of the receptacles, the speed of loading, the length of haul, and means for secondary transportation have secured better economy and efficiency. Small single-horse carts have given way to larger wagons, the tractor-trailer system, and motor trucks. Long hauls have been handled better by secondary methods of transportation, such as trains of tractor-drawn vehicles, railroad cars of special design, barges, and large trucks. Street flushers, snow loaders, sweeping machines, and similar equipment have become available. The displacement of horses by automobiles has lessened the work of street cleaning, and the extension of pavements has made the work done more effective. Thus, a sounder balance between hand and machine work has been secured.

Development in Reduction Plants

The reduction of garbage into grease and fertilizer was first practiced on a working scale in Buffalo, N. Y., in 1887. The Merz process was used, in which the refuse was first dumped into a large hopper from which the free water was allowed to drain. It was then spread out, and cans, bottles, rags, metals, bones, etc., were picked out. The remaining garbage was then passed through hot-air dryers and the partly dried material treated with a solvent for the extraction of grease and the recovery of the solids as tankage or fertilizer base. This process, with some modifications, is still used in Chicago, Ill., Detroit, Mich. (in part), and elsewhere. Other processes for the recovery of grease and fertilizer were invented and plants built. In them the problem of odors and nuisance was difficult to solve.

A notable development in garbage reduction is known as the Cobwell process (1912). The garbage is first placed in a reducer, which is a round, flat-bottomed, covered, steam-jacketed tank, with a diameter of at least twice its height. In this, the garbage is flooded with naphtha and then cooked by admitting steam to the jacket under a pressure of 85 lb. per sq. in. Agitation arms in the reducer keep the contents in motion. The temperature of the garbage and naphtha is held at less than 200° Fahr., under which conditions the original moisture content is carried off with the vaporized solvent. Thus, the garbage is dried. When the water is gone, the temperature in the reducer rises, and the dried garbage is washed several times with naphtha (gasoline) to extract the grease. Finally, the residual material or tankage is dried in the reducer with live steam. Thus the entire process is conducted in a single tank without exposure, and odor control is better secured. Recently, economies in

operation have been made by a partial preliminary drying in hot-air dryers, but with an increased odor hazard.

The greater unit value of the dried product (tankage) as a stock food instead of a fertilizer has led to further developments. At Indianapolis, Ind., Dayton, Ohio, and Michigan City, Ind., so-called "Morrison" plants have been built. In this process the garbage is first placed in vertical digesters or cookers, holding 4 or 5 tons, where it is cooked with live steam for several hours. It is then pressed in the tank under live steam, grease and water being drawn off to a settling tank where the grease is recovered. The tank or digester is then closed tightly and the residue dried further under a partial vacuum by admitting steam to the jacket. After drying, a door near the bottom of the digester is opened, and the dried material is discharged on a conveyor. The dried material or tankage is then separated, screened, and ground for the market. Some grease is recovered by skimming the liquid drained from the digester, but it is understood that a part of it remains in the residual tankage.

At present, following operations at Toledo, Ohio, a so-called "Pan-American" process plant is going into operation at Kansas City, Mo. This process involves a separation of foreign materials, such as metals and glass, from the garbage, with a drying operation. The dried material is then prepared for sale as a stock food.

Except as just stated, developments in garbage reduction plants have been chiefly in better equipment and machinery. Experience gained in operation with dryers, digesters, piping, and other apparatus has resulted in better and more durable design and construction. Materials better able to withstand the corrosive action of the hot liquids and gases are used. Thus, improvements have come in design and construction, rather than in the invention of new processes.

Incineration

Incineration of garbage in the United States was first practiced in 1885 when a plant to burn garbage with coal was designed by Lieut. H. J. Reilly, U. S. A., for the Federal Government, and built on Governors Island, New York. Designs by Engle, Dixon, Decarie, Lewis, Morse, and others, for burning chiefly garbage with an additional fuel followed, and plants were built by several cities. In the meantime, in England and on the Continent, many incinerators or so-called destructors were built for burning a mixture of garbage, ashes, and rubbish, and this was burned generally without additional fuel.

In 1908, J. T. Fetherston, M. Am. Soc. C. E., introduced mixed refuse incineration into America and built plants on Staten Island. Other plants of this type were built in Seattle, Wash., Milwaukee, Wis., Paterson, N. J., Memphis, Tenn., Atlanta, and Savannah, Ga., and elsewhere, generally prior to 1915. In these plants a mixture of garbage, ashes, and rubbish was burned under boilers, and some power was developed. Such works were generally situated nearer to the centers of the cities than was the case with reduction plants, as the odor hazard was less.

Recently, there has been an adaptation of incinerators of this type to the burning of garbage and rubbish only. Experience indicated that ashes added more to the cost of incineration than was justified by their fuel value, and, as they could be dumped satisfactorily, it was more expedient in many cases not to burn them. Adjustments in the design included a wider drying hearth, deeper grates, the use of occasional small quantities of additional fuel, and proper flue areas. Such furnaces operate generally with a forced draft of pre-heated air. Incinerators for this service have been built recently in New York, Brooklyn, N. Y., Philadelphia, Pa., Asheville, N. C., Muskegon, Mich., Charleston, W. Va., and elsewhere.

Perhaps the chief addition to English experience contributed by American designers has been the development of mechanical charging devices and ash-removal apparatus. Many plants are equipped with charging hoppers that are controlled by sliding doors operated by hydraulic pistons and worked by the fireman. The hoppers are filled from above by grab-buckets or by hand from storage hoppers. This arrangement permits 24-hour operation and the use of refuse of such quantity and quality as to maintain the best rate of combustion. A few plants have mechanical clinkering or ash-removal, but, with the omission of ashes from the mixture burned, there has been less need for this device.

Meanwhile, designs have been developed from the older garbage furnaces for burning rubbish with the garbage with a reduced quantity of additional fuel. Such furnaces are of larger internal capacity than the destructor type, and much of the material is discharged directly into them from the collection wagons. Such plants have been built in many Southern cities and in smaller and medium-sized cities of the North. Table 4 contains data indicating the furnace capacities of various types.

TABLE 4.—DATA ON INCINERATOR FURNACES.

(Approximate capacity, 100 tons.)

Type of plant.	AREA, IN SQUARE FEET.		Inside volume of furnace, in cubic feet.	Combustion chamber volume, in cubic feet.	Outside storage hopper, in cubic yards.
	Burning grate.	Drying hearth.			
A	433	192	4 000	1 410	0
B	120	360	3 992	.....	0
C	100	62	976	1 006	290
D	120	80	1 110	924	152
E	181	142	2 296	1 464	150
F	120	....	1 280	2 110	218

Incinerators are still generally purchased by municipalities under general specifications requiring the fulfillment of stated guaranties covering capacities, temperatures of combustion, and operating costs. This practice opens the bidding to a variety of designers, and the selection of a contractor may be decided by a comparison of the total annual costs and the likelihood of satisfactory operation, as judged by records of routine service elsewhere. There is, at present, a tendency toward more detailed specifications and general plans.

Other Methods of Disposal

Perhaps feeding to hogs has changed from early practice more than any other method, although a wide range in operating still prevails. Hogs used to be allowed to roam over dumps (in Louisville as late as 1917) and even streets searching for garbage. Now, better house treatment is practiced, and cleaner, fresher material is hauled to the hogs at one or more farms. It is spread on concrete platforms, and the unconsumed parts and hog manure are gathered and disposed of generally by burial. Shelters and running yards are provided, and the animals are innoculated against cholera, which items have much improved this method of disposal.

Dumps continue to be a chief method for disposing of street sweepings, ashes, trade refuse, some rubbish, and some garbage, and great areas of low and submerged lands near cities have been reclaimed. Parts of Chicago, Boston, and other cities have been converted from low marsh land to drainable upland by filling with refuse. In Seattle, Portland, Ore., and other Pacific Coast cities, garbage has been dumped successfully with a proper cover of street sweepings, waste building materials, and other relatively inert substances. In the smaller cities some garbage is buried. Dumps are better regulated, with less miscellaneous scavenging, better control of fires, and more cover with inert materials. The use of other methods of garbage disposal has influenced the making of better dumps.

Odor Control

The growth of cities and the increased quantity of refuse has emphasized the dangers and discomforts caused by atmospheric pollution. Not only have odors been produced by the special methods of garbage disposal developed, but those from dumps and hog farms have become a nuisance to more people, and hence have demanded attention. Probably no other branch of sanitary engineering causes such frequent recourse to the Courts for the abatement of nuisance. Consequently, measures for odor control and elimination are a mark of progress in this field. Some of the steps which have been taken in America are:

a.—The separation of garbage from other refuse to be disposed of on dumps, so that dumps are more sanitary.

b.—The separation of rubbish from other refuse to prevent fires, smoke, and odors from dumps.

c.—The use of water-tight collection wagons, washed and painted more frequently.

d.—The careful covering of organic matter on dumps with soil or other relatively inert material.

e.—The use of concrete feeding platforms at hog farms, and the better disposal of residues.

f.—Not only the use of incinerators, but improved design and construction, including better facilities for receiving and handling refuse at the plant, and more ample combustion chambers.

g.—Chlorination, scrubbing, and other improved methods of treating dryer and digester gases from reduction plants.

h.—Better floors and buildings, and an appreciation of the advantages of general plant cleanliness.

i.—Careful selection of disposal plant locations, with reference to the cost of delivering refuse and the character of plant.

Summary

The cleansing of cities, including street cleaning and the collection and disposal of refuse, has thus progressed during the past 150 years with their growth and the development of sanitary science. During the first half of the period, there were few large cities, and the problem was not acute. Following the middle of the Nineteenth Century, cities enlarged rapidly and required cleansing. A full realization of the dangers of incomplete city cleansing, with means for its better accomplishment, did not appear, however, until after the Centennial year. Since that time the dangers and discomforts to the public from filth and dirt have been recognized, and sanitary engineering has made use of available appliances and has developed methods for the cleaning of streets, the collection of refuse, and its final disposal. The operation of these appliances and methods has called for skilled management, and the employment and organization by public officials of a considerable number of men. Consequently, the efficiency with which appliances have been used and the excellence of the work done has depended very largely on the quality of the public administration. The cleansing of cities is thus interwoven so clearly with their management that efficiency of operation and progress in the future will depend in large measure on the character and training of municipal governments.

HISTORICAL NOTES ON LAND DRAINAGE IN THE UNITED STATES

By S. H. McCrory,* M. Am. Soc. C. E.

Land drainage in the past 150 years has had an important part in the development of American agriculture and in the improvement of the public health. Complete statistics of drainage have not been compiled, but 50 000 000 acres added to the cultivable land area in the United States and an equal amount made more productive and profitable doubtless would be a conservative estimate of the benefit to agriculture. The benefits to health would be exceedingly difficult to measure, but it is known from early records that malaria was generally prevalent in Illinois, Indiana, Michigan, and Ohio, where drainage is most complete, and that it is much less common in those States to-day. The experience in the South, also, has been that after drainage the extent and seriousness of malaria have generally diminished.

Early Drainage in America

Drainage of farm lands undoubtedly began in each of the Colonies soon after the first settlement was made. Early practices followed those in use in England at that time. Open ditches probably were most used, but blind drains made by digging a trench and filling the lower part with poles, brush, or stones, were used to some extent. In 1767, Elkington, of England, discovered his method of intercepting drainage; this was described in a treatise by John Johnston in 1797, and laid the foundation for modern under-drainage. It was reprinted in England several times, and in 1838 was first published in the United States at Petersburg, Va.† In 1804, drain tile was laid in England; perhaps James Graham was the first user, and tile laid by him were still giving good service in 1887. Deep, or "thorough", drainage was introduced in England by James Smith, of Devonston, Sterlingshire, Scotland; his views came under general notice about 1832, and had great influence on drainage work in England.

In America, under-drainage started in 1835 when John Johnston, of Seneca County, New York, brought over from Scotland patterns from which clay tiles were moulded by hand and laid on his farm near Geneva. By 1838, hand-made tiles were manufactured at Waterford, N. Y., and sold for $24 per 1 000 pieces. In 1851, Mr. Johnston reported that he had laid 16 miles of tile drains on his farm; and in the meantime a number of other farmers, chiefly in New York State, had followed his example and reported success. Among these American pioneers in the art of draining was John Delafield, a neighbor of Johnston's, who in 1848 imported what was believed to be the first tile-making machine used in America. This operated on the same principle as

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† Supplement to the *Farmers' Register*.

the modern machines for making clay tile, and the work it could do has been described,* as follows:

"This improved tile machine produces drain tiles in the form of a horseshoe, rising 2½ and 4 inches; it makes pipes also for draining, from 1 inch bore up to 6 inches diameter, each size varying about half an inch. Soles or flat tiles are made on which to lay the tiles when desired. Semi-cylinders are made of 8 and 11 inches diameter. It also produces with equal readiness, pan tiles for roofing and ridge tiles."

The horseshoe-shaped tiles, 12 to 15 in. long and usually laid on the flat sole tiles, were the common kind in the beginning. The cylindrical tiles do not seem to have come much into use for drainage until about 1858.

The principles underlying successful under-drainage apparently were well understood by Johnston and his contemporary progressive farmers, for one finds, in the discussions and papers of that day, the current arguments advanced for under-drainage, such as soil aeration and warming, improvement of soil texture, and removal of injurious salts. Moreover, stress is laid on the necessity for "thorough" drainage, as contrasted with shallow drainage. As early as 1848 John Delafield stressed the value of drainage for much land that was already sufficiently dry to be producing cultivated crops.

Development of Tile Drainage

Stone and pole drains were still much in vogue after the middle of the Nineteenth Century, but perhaps no one since has better stated the present view as to the superiority of tile over these materials than did Henry D. French when he said:† "No system of drainage can be made sufficiently cheap and efficient for general adoption, with other materials than drain-tiles."

Modern drainage began in the United States when Johnston laid the first tile drains, in 1835. The Ohio and Mississippi Valleys were then just being settled. Much of this land, although very fertile, could not be cultivated until it was drained and malaria was prevalent throughout large sections. Following the importation of the first tile-making machine, in 1848, the use of tile for drainage spread rapidly. In 1853, Samuel Lillie, of Allen County, made the first tile in Indiana, and, in 1859, James Stewart started the first horse-power tile factory in that State. The number of tile factories established in the United States were as follows: from 1850 to 1859 inclusive, 66; from 1860 to 1869, 234; and from 1870 to 1879, 840.

On January 1, 1880, there were 1 140 factories in operation, as follows: Indiana, 486; Illinois, 320; Ohio, 230; Michigan, 63; Iowa, 18; Wisconsin, 13; New York, 8; and Pennsylvania 2. Thus, at that date, the center of tile manufacture had already moved from the East to the Middle West. To May 1, 1882, there had been laid in Indiana, 9 798 256 rods (30 619.55 miles) of tile, with one county not included in the statistics.

The sizes manufactured increased steadily, and in the early Eighties some plants were able to make 24-in. tile. Machines for manufacturing concrete tile were placed on the market in the Eighties, and the use of this material

* *Transactions*, New York State Agricultural Soc., 1848.

† In his book "Farm Drainage," 1859.

increased gradually especially, for tile 18 in. in diameter or larger. The sizes used in farm drainage have increased gradually; very little tile smaller than 4 in. is now used for drainage, and in large sections of the Middle West, 5-in. tile is considered the minimum size. In Iowa and Minnesota, 36-in. tile outlet drains are now common and drains from 42 to 54 in. in diameter are constructed occasionally.

The hand tools used in constructing drains underwent much improvement during the period from 1840 to 1880. About 1873, Manley Miles invented a drain scoop which could be either pushed or pulled, and made grading the trench much more rapid and easy than it had been. The skeleton spade was also invented during this period.

As the sizes of tiles and the depth of drains increased, there came a demand for a machine that would dig trenches. Among the early types were the Blickenderfer ditching machine, the Hickok, the Rennie elevator ditcher (patented in 1869), and the Johnson tile ditcher, all operated by horses. In 1880, seven kinds of ditching machines were reported* in use, or as having been tested, but with unsatisfactory results. In 1883, the Plumb ditching machine was demonstrated. It was the first one operated by steam, and seems to have been fairly successful. Soon after this several other steam-operated machines came on the market, and from these the modern trenching machines were developed.

The problem of computing the carrying capacity of drain tile received serious consideration by the early drainage engineers. Poncelet's formulas were generally used. In 1916 and 1917, Yarnell conducted extensive experiments and developed formulas for the flow in drain tile.

Investigations of the run-off from drained areas and the relation of depth and spacing of tile to drainage, have been or are in progress in Alabama, Iowa, Illinois, North Carolina, and Minnesota. The problem is intricate and much additional work will be necessary before reaching a full understanding of the laws that control the movements of water to drains.

Construction of Open Ditches

It early became apparent that if under-drains were to be successful, large outlet drains must be provided. These were mostly open ditches, constructed either by hand, or with teams and scrapers. Usually they were not more than 5 ft. deep, the bottom width only occasionally exceeding 4 ft. In Ohio, Indiana, and Michigan, and in the Upper Mississippi Valley States, the construction of such outlet drains increased rapidly as the States were settled. In January, 1884, the Drainage Committee of the Ohio Society of Engineers and Surveyors reported that, in Ohio, 20 000 miles of county and township ditches had been constructed, benefiting 11 000 000 acres of land. The report placed much emphasis on the benefit these drains had been to the health of the State. Most of these ditches were found to be too small, however, for providing the degree of drainage that was desired. There was need for an economical means of constructing large open ditches.

* *The Drainage Journal*, 1880.

In 1882 the Mason and Tazewell Special Drainage District was organized in Mason and Tazewell Counties, Illinois. The drainage plan of this district comprised a main ditch 14½ miles long, 10 ft. deep, varying in top width from 30 to 60 ft., with side slopes of 1 to 1; approximately 37 miles of lateral ditches, 10 feet deep; and 30 miles of spade and scraper ditches. About 7 miles of laterals were added later. Construction was started in 1883 and completed in 1886. The cost of the improvement was approximately $300 000. On this work five dipper dredges were used by the contractor, W. A. McGillis and Company. The dredges were constructed by the Bucyrus Foundry and Machinery Company, of Bucyrus, Ohio, and probably were patterned after dredges used in digging navigation canals. This was the first drainage district on which dredges were used, and the main ditch when constructed was the longest in the United States. Mr. Herbert Hamilton, of Petersburg, Ill., was the Engineer for the District. The *Engineering News*, in commenting on the work, stated: "This was the first work done with a dredge, and has been so satisfactory that it may be said to have worked a revolution in such matters."

Since this work was done the development of dredges has been rapid. The vertical spud was replaced by the bank spud, which permitted the use of a narrower hull, thus enabling the machine to dig smaller ditches. On the other hand, the design of larger and larger ditches led to the design of larger dredges, until at present (1926) floating dredges range in size from the small ½-yd. machine, with a 25- or 30-ft. boom, to the 5-yd. machine with a 100- to 125-ft. boom. The success of floating dredges led to the development of the dry-land dipper excavator for use where sufficient water was not obtainable for the floating type.

Soon after the first dipper-dredges were put in operation, dredges equipped with grab-buckets were used, their principal advantage being that they could be equipped with a longer boom. Machines of this type have been highly developed in California, where they are used for ditches and levees in peat lands.

In 1903 Page and Schnable built the first drag-scraper excavator, and this was used on the Hennepin Canal. It was first used on ditches in 1906, and was originally considered a dry-land excavator, but it has gradually supplanted the floating dipper-dredge for constructing drainage ditches. It is built in a wide variety of sizes, and various methods are used for moving it. The great advantage of the drag-line excavator over other types is its flexibility. There are well-defined limits to the size of ditches that can be constructed by floating dipper-dredges of a given size, but such limits do not apply so closely to the drag-line. It has the added advantage that on large ditches it is possible to provide a wider and cleaner berm than is possible with dipper excavators. Experience has shown that the securing of an ample berm is of much importance when large deep ditches are to be dug.

The early dredges were operated by steam engines fired with either wood or coal. Internal combustion engines began to be used on drag-line excavators soon after their introduction. Electric current, where readily available, now affords a convenient source of power.

Pumping for Drainage

The use of low-lift pumps to drain lands was early practiced along the coast. Drainage wheels were first used, and were operated probably by animal power or by windmills. About 1850 centrifugal pumps began to be used. In the late Sixties the Ivens pump was invented, and special designs for low-lift pumping were developed and used extensively. In 1880 the Menge pump was introduced. It could be easily and cheaply erected, and was used widely for drainage pumping.

As projects became larger, and centrifugal pumps were improved in efficiency, they gradually supplanted other types. Recently, screw pumps, giving still higher efficiency on low lifts, have been introduced. About 1900 drainage by pumping began to be practiced along the Upper Mississippi and Illinois Rivers. At present (1926), practically all the bottom-land along the Illinois River has been reclaimed in this way.

Drainage of Irrigated Land

Soon after irrigation was started in the West, some of the land began to be damaged by seepage and the accumulation of alkalies. As early as 1884, Mr. George C. Roeding, a nurseryman of Fresno, Calif., began drainage experiments to relieve this condition. He laid tile drains in 1893 which were said to be still working in 1909. As this trouble was becoming widespread, the Bureau of Soils, U. S. Department of Agriculture, built several experimental drainage systems in 1902. These experimental systems followed Eastern drainage practices as to depth and opening, but were not as effective as had been expected. In 1902 the position of Drainage Expert was created in the U. S. Department of Agriculture, and the late C. G. Elliott, M. Am. Soc. C. E., a prominent drainage engineer, was appointed. He gave much attention to the drainage of irrigated lands; and a small group of engineers was assembled to work on this problem It was found that shallow drains were ineffective, but that a deep drain, properly located, frequently would drain an entire field. From the work thus started has developed a method of drainage which has reclaimed or protected much irrigated land.

Lands containing calcium carbonate—commonly, called black alkali—have always been exceedingly difficult to reclaim by drainage. Recently, chemists as well as engineers have been devoting considerable study to this problem, but it has not yet been solved. Seepage and the accumulation of alkalies must be prevented if irrigation agriculture is to be made permanent. The engineers of the United States have developed and perfected methods of draining irrigated land to a greater extent than ever before.

Drainage District Laws

Even in small drainage undertakings several land owners frequently are interested, and it is often difficult to secure voluntary co-operation. When many land owners are concerned, it is practically impossible to secure unanimous action. This difficulty brought about the passage of laws which provide means of constructing necessary drains and forcing all land owners to pay their just share of the cost.

Almost every State now has a general law which permits the organization of drainage districts, provides the necessary means of securing co-operation among a number of land owners to construct the improvement, and specifies the manner of distributing the costs among the affected lands, and of securing the privileges of longtime payments. These laws are based on early English laws for drainage and sewerage, which provide that a citizen shall consent to pay his proportional part of the cost of a public, local improvement in the same way that he consents to share in other expenses of government. The authority to enact such laws is exercised by the State Legislatures, and is subject to the limitations placed on legislation by State and Federal Constitutions. The history of drainage legislation in each State is a story of repeated attempts to enact laws which should conform to these constitutional requirements, and meet the changing needs of the owners of wet or overflowed lands.

The first drainage law in the United States of which there is authentic record was enacted by the Colony of New Jersey, on September 26, 1772, and it was re-enacted by the State when it adopted its Constitution, on July 2, 1776. This was an act drawn for the purpose of clearing obstructions in the free course of the waters of the Passiac River, near Little Falls.

The act provided for co-operation among a number of land owners to secure field protection, for a division of the costs, and for a method of collecting the costs; it established the public interest in such work by reason of the improvement of the public highways. It did not, however, provide for the inclusion and assessment of a minority of land owners who did not desire to co-operate in the project, nor did it provide for spreading the payments over a period of years.

One of the earliest general drainage acts was passed on April 9, 1804, in New York State. This act was declared unconstitutional, on the ground that it provided for the taking of private property for a private, rather than a public, use.

Another early general drainage law is found in the Revised Statutes of Massachusetts, 1836, Chapter 115. This act provided for the improvement of low lands by removing obstructions from streams, and by ditching. It provided for a petition to be signed by a majority of land owners setting forth the proposed improvement, for notice to land owners who did not sign the petition, for a judicial hearing, for a board of commissioners to be appointed by the Court to construct the improvements, for an assessment of the expenses to be made on all proprietors according to the benefit each should receive from it, for damages to be paid to injured parties, and for appeals to high Courts. Thus it is clear that this law was a marked advance over earlier ones in that it provided for the forced participation of non-petitioning land owners and for legal safeguards to prevent abuses of this power.

Under the earlier drainage acts the work to be done was small, and in many cases was performed by the land owners themselves, but, as the projects grew in size, and expensive machinery was required for economical construction, it was necessary to engage contractors, and these contractors had to be paid on the completion of the work. As it was impossible for the land

owners to raise the large sums thus required, drainage development, for a time, awaited, legislation which would allow adequate and proper financing. In 1879, Illinois passed its Levee and Farm Drainage Acts, which permitted the district to issue bonds to be sold for cash to pay for the construction. These bonds were first liens on all assessed lands within the district, and arrangements were made whereby the land owners had a long time in which to pay the costs assessed against them and thus redeem the bonds.

Except for minor improvements and adjustments further to protect the interests of the bond buyers, this was the final step in the development of general drainage laws as now known. In almost every State laws have been developed, and approved by the Courts, by which drainage and flood protection districts can be established and financed under the supervision of the Courts, having the following characteristics:

1.—By law they are made a continuing form of corporation, and act through a board of officers.

2.—They can borrow money and issue bonds in the name of the district.

3.—They are financially responsible, and may sue and be sued in the district's name.

4.—They possess machinery for making an equitable distribution of costs in proportion to benefits conferred.

5.—They are authorized to levy taxes against each piece of land, to the extent of benefits conferred, and to use State machinery for collecting such taxes.

6.—They have the power of eminent domain, which gives them the right to condemn property needed for carrying out the purposes of the district.

7.—They possess the power to construct such improvements as are necessary to accomplish the reclamation of the lands in the district.

8.—They may hold and convey property.

Extent and Cost of Drainage

Statistics as to the growth of drainage by private individuals, firms, or corporations, have not been collected, but, in 1920, the Fourteenth Decennial Census gathered data as to the status of drainage on farms. These figures show that about 53 000 000 acres were drained by ditches or tile. This is equivalent to 10.5% of the improved lands in farms. Illinois has drained more than 8 000 000; Iowa and Ohio more than 7 000 000 acres. Farm lands still needing drainage were reported as 39 000 000 acres.

During the Census of 1920 statistics concerning land drained by drainage districts were also collected. In 1920, the area benefited and assessed for the cost of the drainage thus provided was more than 65 500 000 acres. These figures do not include statistics for levee districts which had not constructed drains. In addition, 3 900 000 acres were in organized drainage districts which had not begun construction. The area is probably not less than 75 000 000 acres in 1926, estimating conservatively from nearly complete data secured in 1923. There has been much overlapping of enterprises, particularly in Ohio, Michigan, and Indiana, chiefly because in some States the enlargement or clearing of an old drain required the establishment of a new drainage district.

According to the Census, the drainage works of the districts completed or under construction prior to 1920 comprise 111 770 miles of open ditch, 45 174 miles of tile drains, and 4 330 miles of levees, or a total of 161 274 miles. All but 7 974 miles had been completed on the Census date. The cost of this drainage cannot be determined accurately as records of the early work are incomplete in States where the most has been done by allotment to the land owner instead of by contract. The Census shows $183 000 as the cost of all enterprises prior to 1860; $10 000 000 to 1880; $60 800 000 to 1900; and $434 600 000 to 1920, including the cost of the completion of work under construction. The cost of drainage done by farmers is difficult to estimate. From fragmentary data available it is conservatively estimated that at least $600 000 000 has been spent for such improvements, and, if more complete information were available, it is possible that the figure might reach $1 000 000 000. Drainage work by districts and individuals had cost, to 1920, from $1 150 000 000 to $1 500 000 000.

What have these works accomplished? First, they have generally greatly improved the health of the communities in which they were constructed; second, they have made available to the farmers about 65 000 000 acres of the best land in the United States—land that, in its original condition, could not be cultivated profitably; third, they have increased yields, and made crops more certain on the lands drained; fourth, they have reduced the losses of crops due to floods and inadequate drainage; and fifth, they have removed surplus water and thus made easier the construction and maintenance of roads and railroads.

Future Developments in Drainage

In the future benefits accruing to agriculture and health will cause much land to be underdrained that at present is not thought to need it. Within the next two decades the Middle, South Atlantic, and Gulf States will be practicing under-drainage on the same scale as now followed in Iowa, Illinois, and Indiana.

As the density of population increases and more intensive farming is practiced, much land now drained will need more thorough drainage. This will require deeper and larger outlet ditches, with flatter side slopes than has been customary in the past; and detention reservoirs, to prevent damage by floods, will become practicable in many places where they are not now economically feasible.

Excavators capable of digging, without rehandling, ditches with a cross-sectional area of 6 000 sq. ft. and upward, to any desired section, with berms of any width desired, and capable of distributing the excavated material in such manner that the weight will not be unduly concentrated, will be developed.

Tidal marshes will be reclaimed. This will involve construction of embankments and pumping plants of a magnitude not yet attempted, particularly along the Gulf Coast, where, in addition to securing drainage, it will be necessary to provide protection from storm tides caused by tropical hurricanes.

The size of drainage districts will increase, and eventually districts will be organized co-extensive with the water-sheds on streams, such as the Saginaw in Michigan, or the Yazoo in Mississippi, with permanent staffs who will handle all problems involving water, such as drainage, flood control, water power, navigation, and control of erosion.

Drainage laws will be further simplified and made more effective. Simpler and less costly methods of organizing small districts will be developed. It is to be hoped that the administration of districts will eventually be entirely removed from the judiciary and assigned to men qualified by training and experience for such work.

HISTORICAL REVIEW OF DEVELOPMENT OF CONTROL OF DISEASE-BEARING MOSQUITOES

By J. A. LePrince,* Esq.

Disease-bearing mosquitoes have played an important part in the early settlement and development of all countries in the temperate and tropical zones of the Western Hemisphere. The second expedition of Columbus was nearly wrecked by them at Santo Domingo, and, later, Prescott wrote of mosquito-borne disease as follows: "The moment a town is founded, or a commercial center created, it is certain to cause the explosion of the latent malignity of poison in the air." In early days the Colonies suffered severely from both yellow fever and malaria. Quinine was scarce and costly, and other remedies used for malaria had little value.

Mosquito-Borne Disease a Menace of Importance

At present, one is apt to consider yellow fever as a semi-tropical, or tropical, seacoast disease, but it has not always been so. It was intermittently present in the Southern States from Colonial days to 1905, and not entirely absent in the Northern States. In eight States north of Virginia, from 1693 to 1870, eighty yellow fever outbreaks are recorded.

In the records of New York, N. Y., yellow fever appeared during seventeen summers, and at Philadelphia, Pa., during fifteen summers. Reports show 2 086 deaths from yellow fever in New York in 1798, and 3 506 in Philadelphia in the same season. In the Philadelphia epidemic of 1793, in a population of about 40 000, more than 4 000 deaths were recorded, and reporting was not as complete then as it is to-day. Longfellow's "Evangeline" contains a description of one of these Philadelphia outbreaks.

The Colonists were painfully aware that malaria played a decidedly important part in retarding development. Jamestown, Va., was situated on low land close to a large wet depression. No attempt was made at drainage, and it is easy, to-day, to understand why this Colony was a failure. It is recorded that the original colony at Charleston, S. C., was first situated on the mainland near swampy areas like those around Jamestown, and two years later, due to sickness among the colonists, was moved to its present location, at the end of a peninsula, where health conditions were more satisfactory. In the early days at Savannah, Ga., malignant malaria fever was common, and the situation became so serious that the town authorities passed a law to forbid the maintenance of "wet culture," as rice was then called, and reimbursed land owners for loss of potential crops. This was probably the first serious early attempt at malaria reduction by an American city.

Following the Declaration of Independence, with the settling of new territory westward from the Atlantic Coastal Plain, malaria prevalence was common, even as far north as New England, Illinois, and Michigan. In fact, it

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was widely distributed in the Northern States as late as the Eighties, and then began to decrease with the extension of intensive farm drainage, improved living conditions, the reduction in the cost of quinine and its more general use, and the provision of metallic screens for the protecton of homes.

During the Civil War one-fourth of all hospital admissions were due to malaria fever, and in military camps in the Southern States in the Spanish-American War period so many soldiers were infected by malaria before leaving for Cuba that only a small part of some regiments was fit for service.

The First Mosquito Campaign

During the Nineteenth Century the number of deaths from malaria among the native population of Havana, Cuba, in some years was greater than that of the total deaths caused by yellow fever. The first effective campaign against mosquito-borne disease was begun by Americans at Havana in 1901. At that time yellow fever had become so prevalent in that city as to affect seriously the military occupation of the Island. The cause of the transmission of the disease was then discovered; the mosquito that conveyed it was reduced below the danger point, and yellow fever was eliminated.

During the first yellow fever battle the campaigners felt so certain of the final outcome that they instituted an anti-malaria campaign, and reduced malaria prevalence about 80% before the first period of American occupation terminated. A few years later, due to the absence of yellow fever and the rarity of malaria, the city became a tourist resort, although previous to anti-mosquito measures, it was known as the cradle of yellow fever and malaria.

It is of interest to know that, during this historic campaign, an American sanitary engineer kept a chart, in connection with yellow fever control measures, and estimated the date when yellow fever would be eradicated. As he happened to be directing the field control measures, he became too enthusiastic over obtaining results, and yellow fever was eradicated 60 days earlier than his first estimate. His friends having told him that an engineer's estimate should be more accurate, he replied that it would be made so as soon as they could show him how to determine the existence and location of walking cases of yellow fever.

Although it was thus definitely proved in 1901 that both yellow fever and malaria could be eliminated at a reasonable cost, no serious and thorough effort was made to control disease-bearing mosquitoes in the United States until the outbreak of yellow fever in New Orleans, La., in 1905 and then only in that city until yellow fever had been eliminated.

The Panama Canal Zone was occupied by Americans in 1904; yellow fever was eliminated by 1905, and an extensive anti-malaria campaign was initiated and has since been continued.

English Suggestions and American Inventions

The mode of transmission of malaria was discovered by Sir Ronald Ross, who, during his visit to Panama in 1905, stated:

"What is most necessary for *Anopheles* control is for us to find a cheap powder to act as a larvacide that we can toss to the right or left, and also we should devise a trap we can bait to catch disease-bearing mosquitoes."

These two suggestions were developed later by American sanitarians. One of the most prolific sources of malaria mosquitoes known is on the Potomac River, and is now being treated by the application of dust clouds of Paris green from aeroplanes; and the mosquito trap has been perfected so that one trap, less than 2 cu. ft. in volume, has caught more than 1 100 malaria mosquitoes in a single night. The inventor used for bait the odor of the canal construction laborers whom these mosquitoes desired to bite.

The Public and a New Idea

It is surprising that no attempt was made to control disease-bearing mosquitoes in the United States until 1913, when malaria surveys were first made in Virginia and North Carolina by Dr. H. R. Carter, of the United States Public Health Service, on his return from Panama. Dr. Carter was a graduate civil engineer as well as a physician of international reputation, and, because of his education and experience as a public health officer in the control of yellow fever and malaria, was peculiarly fitted to define the malaria problem of the United States and outline a program for its solution. At that time it was estimated that the annual economic loss caused by malaria alone in this country was $100 000 000. The ravages of yellow fever in Southern ports had also caused many deaths, much suffering, and enormous financial losses. Even as late as 1922, when a dengue epidemic swept over the country from Florida to Texas, its rapid rate of travel indicated clearly the extensive prevalence of the yellow fever mosquito in towns and villages. This mosquito spreads both yellow fever and dengue, and breeds only in man-made water containers in or near occupied homes.

Malaria Control in Military Camps

In 1914, a Federal appropriation of $16 000 was made for malaria studies and investigations by the U. S. Public Health Service, and in conducting these investigational studies 22 surveys and 3 malaria control demonstrations were made in 7 States during that year. This appropriation and the investigational study were continued until the United States entered the World War, when the small personnel employed were assigned to control measures connected with war activities. At that time zones of health protection were established at all cantonment towns and in areas a mile wide surrounding each military cantonment and near-by town in all sections where malaria was prevalent.

If the same malaria sick rate and death rate that took place in Southern military camps during the Spanish-American War had prevailed during the World War, there would probably have been a loss of 5 000 men by deaths from malaria. By rapid training of engineers in malaria-control measures, and quick, hard work, the total deaths from malaria in Southern military camps throughout the war was held down to 36. This enormous reduction is worthy of consideration, yet more important is the fact that enlisted men were not sent to France saturated with malaria to unfit for duty a large part of the military forces, as was the case in the Spanish-American War before the method of malaria transmission and control measures were known. During the World War, 193 men were constantly incapacitated by malaria, but with the same

rate as obtained in the Civil and Spanish-American Wars, 38 000 would have been incapacitated each day of the war from April, 1917, to December, 1919.

At the termination of the World War, the U. S. Public Health Service co-operated with the International Health Board and the State Health Departments of Southern States, and succeeded in starting malaria-control activities in fourteen of such States. As soon as the State Health Departments were able to supervise their local control projects, Federal supervision was withdrawn, and, to-day, the State Health Departments are assisting counties and communities in mosquito-control activities.

Previous to 1914 probably less than six communities had undertaken malaria-control measures, and the results of such limited effort were unsatisfactory. Up to that time, not a single State health department was taking any action toward malaria elimination, although in many counties malaria was the most important health problem. No State or county had made any appropriation for malaria reduction. In 1916, the physicians of one State alone reported to the State Health Department 158 779 cases of malaria.

Public Demand for Mosquito Elimination

As an illustration of the change in the public attitude toward malaria control and mosquito elimination that has taken place since 1914, the U. S. Public Health Service has co-operated with State Health Departments in malaria-control measures in 343 communities in 216 counties in 17 States, and in activities connected with mosquito surveys, control measures, and investigations, in 667 communities in 376 counties in 24 States.

In addition, many communities have undertaken control measures that are not included in the recorded data, and the State Health Departments of Alabama, Georgia, and Texas, report that they are now overwhelmed with requests for advisory supervision from communities desiring to undertake mosquito-control activities.

It was the writer's good fortune to be present when the campaign against disease-bearing mosquitoes in the Western Hemisphere was inaugurated at Havana, Cuba, in 1901, to take part in the sanitation of that city, and then go with General Gorgas and Dr. Carter to the Canal Zone, where the fight against disease-bearing mosquitoes was inaugurated under even greater difficulties, but with equal success. In 1914, the writer returned to the United States, and since that time has followed the trail of the *Anopheles* all over the South.

As soon as the way was found, it was comparatively easy to control *Stegomyia* mosquitoes and eradicate yellow fever. The control of *Anopheles* and malaria is a much more complicated matter, but it can be done. Yellow fever has not been of any sanitary importance to any city in the United States for the past 20 years, and within that time malaria has been greatly reduced. Who can say what the next 25 years will bring forth! Every indication points to a continued reduction in malaria throughout the United States at an equal or possibly even more rapid rate than has been attained in the past 25 years. Should this prove true, it needs no prophet to foretell the outcome.

CHANGING CONCEPTIONS OF VENTILATION SINCE THE EIGHTEENTH CENTURY

By Dr. George Truman Palmer.*

The history of ventilation is a chronicle of the effort to analyze accurately the causes of a harmful effect. The effect has been human discomfort, lowered efficiency, and, not infrequently, injury to health.

In general terms, the cause has been bad ventilation. For years, however, there has been difficulty in picking out the specific features that make for badness, and this has handicapped the engineer in designing proper corrective measures.

At the birth of the American Republic there was a consciousness that the air of an enclosure occupied by human beings had some real though intangible relation to health. It was the growing concentration of population, however, which really brought ventilation to the fore. Increasing numbers of children in school, the substitution of the factory system for isolated home manufacturing, the advent of many-storied dwellings and office buildings, the construction of commodious theatres and other meeting places, all these conditions have accentuated the necessity of lessening the discomfort inevitable from indoor crowding.

Vast changes have taken place, both in understanding the basic principles of ventilation and in methods of adapting indoor air to healthful uses.

A century and a half ago it took the most persuasive arguments of Benjamin Franklin to secure the opening of a sleeping-room window when he and John Adams had occasion to be bedfellows in an hostelry in New Brunswick, N. J. To-day, the open bedroom window is general practice.

The grate and the stove warmed habitations prior to 1800. To-day, a steam pipe serves this purpose, even if one's office may be on the fiftieth floor of a skyscraper.

In 1834 ventilation of the British House of Parliament was served by a tower on the roof, 110 ft. high, with a fire at the base for the purpose of creating a draft; and this pulled air into the room through thousands of small holes in the floor.

To-day, not only public buildings, but factories, theatres, eating places, and other inclosures are served by smoothly running direct-connected, electrically-driven, multi-vane blowers controlled by an insignificant looking switch.

The Unfolding of Ventilation Principles

Ventilation has had a more difficult course to pursue than bridge building. In addition to the laws of gravity, stresses, and strains, ventilation has had to satisfy the shifting theories of the hygienist and, worst of all, the discordant demands of human comfort.

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Several things happen to the air in a crowded room: The temperature rises; humidity increases; oxygen diminishes; carbon dioxide increases; volatile organic matter and unpleasant odors are added; more dust is afloat and, largely in consequence, more floating bacteria can be detected.

Are all these changes identified with discomfort? If not, which are important and which are not? The job of the ventilating engineer is to restore comfort. Against what shall he direct his attack? It has taken a century and a half to tag these elements, and even to-day it can hardly be said that complete satisfaction has been attained.

The study of these ventilation factors began in the chemical laboratory. Prior to 1880, three main theories prevailed as to the causes of discomfort in a crowded room: First, that it was due to a diminution in oxygen; second, that it was caused by the poisoning effect of the accumulating carbon dioxide; and, third, that it was due to the toxic effect of organic matter given off in the breath.

The early suspicion against oxygen depletion arose from the then recent discovery (1774) of the existence and significance of oxygen in respiration. It was assumed that the amount of depletion was serious. It is now known that, owing to the multiple opportunities for leakage through walls and crevices, the oxygen content is actually lowered only an insignificant amount. In fact, the worst ventilated room has far more oxygen than a mountain health resort where the partial pressure of oxygen is greatly reduced by the high elevation.

It was Lavoisier who shifted the suspicion from oxygen to carbon dioxide. There was very little basis for this view, but, once enunciated, it has persisted even to this day. Brown, in 1870, refers to carbon dioxide as a "fatal gas". Even in 1913 the engineers, Hoffman and Raber, in their Handbook, class carbon dioxide with the other "impurities", and regret that some way has not been found for dissociating it from the remainder of the atmosphere.

Pettenkofer, in 1863, made it clear that carbon dioxide was not in itself poisonous, but that it served as a useful index of other harmful substances in respired air which he believed were instrumental in lowering resistance to disease. This suggestion gave birth to the carbon-dioxide standard of ventilation, and this, in turn, led to the most common air-flow standard of 30 cu. ft. per min. per person.

The worst ventilated room rarely reaches 50 parts of carbon dioxide in 10 000. Before unfavorable symptoms can be produced, under laboratory conditions, it is necessary for the carbon dioxide to be increased to 300 or 400 parts. The school child actually breathes more carbon dioxide when curled up on his sleeping porch than when sitting in the school room.

The organic-poison theory was sponsored by Brown-Sequard and d'Arsonval. Their viewpoint is represented by Russell in the "Smithsonian Miscellaneous Collections", who wrote as late as 1896, that organic matter in respired air "is a dangerous and pernicious element in all aggregations and, combined with carbon dioxide, produces, when in moderate quantity, depression, headache, sickness, and other ailments." The great majority of workers have

been quite unable to produce the slightest evidence of the existence of an organic poison in respired air.

The relative unimportance of the chemical factors in the air as a cause of discomfort and the primary significance of the thermal factors—temperature, moisture, and air motion—was first brought out clearly by Hermans in 1883. In 1896, Billings, Mitchell, and Bergy in America reviewed and repeated much of the experimental work previously reported. They voiced the conclusion that it is "very improbable that the minute quantity of organic matter contained in the air expired from human beings has any deleterious influence on men who inhale it in ordinary rooms, and, hence, it is probably unnecessary to take this factor into account in providing for the ventilation of such rooms". In their eyes, the two great causes of discomfort, although not the only ones, in crowded, ill-ventilated rooms, were excessive temperature and odors.

This viewpoint has been confirmed in substance again and again by Flügge and his associates in Germany, by Haldane, Pembrey, Leonard Hill, and others in England, and by Benedict, Crowder, the New York State Commission on Ventilation, and others in America.

Present-Day Conceptions of Ventilation

The present-day attitude with respect to the different elements in ventilation may be summed up as follows:

Elevated indoor temperature in the winter, even up to 75°, produces discomfort, lowers working efficiency, forces the heart to extra effort, and measurably elevates the body temperature. As far as possible, the air of an inclosure should be cool and fluctuating in temperature within narrow limits, rather than constant or unvarying.

Likewise, air motion should be variable or fitful, rather than of uniform velocity. Variability or pulsation in air motion stimulates the pleasant feeling of an outdoor breeze, cooling, but not sufficiently prolonged to be chilling.

Humidity may vary within rather wide limits and still be comfortable. In fact, there is little to justify humidities above 40% in the ordinary room in winter at temperatures around 70 degrees. Nor is there evidence that humidities as low as 20% are harmful or uncomfortable. There is a difference between dryness and "dryingness". A low humidity without marked air velocity may be dry, but not drying.

The present air-flow standard is undoubtedly excessive. This may be decreased by one-half in such places as school rooms without discomfort or harm to the occupants.

Except with very dirty clothing and people, there will not be volatilization of odors in cool rooms. One objection to higher humidities is that it accentuates odor.

Ordinarily, the bacterial content of indoor air is of no practical importance. The likelihood of disease being spread by bacteria that float about in the air is so remote as to be beneath serious consideration. In the ordinary room the dust content of the atmosphere is inconsequential to health.

Organic matter and carbon dioxide need not enter into the calculation, although measurements of the latter are useful as an indirect determination of the amount of air flow.

It is recognized, of course, that in special situations factors ordinarily unimportant loom large. In the submarine, oxygen depletion and excessive carbon dioxide are factors to be reckoned with. In certain industrial processes dust or fume removal is of the highest importance; and some situations require high humidity, as in the textile industry.

Changing Practices in Ventilation

Although theories relating to ventilation have shifted from time to time, their influence on practice has not been felt until recently. This is because no deliberate effort was made a century ago to ventilate the ordinarily occupied room. Efforts, to be sure, were made in special fields, such as the rotary fan for deep mines, used in Savoy and Bohemia, as far back as 1553, and the use of bellows for extracting air from the holds of ships, devised by Triewald, the Swedish architect, in 1741. Also, there is the application of bellows for hospitals and mines, described by Stephen Hales, an English clergyman, in 1758. Prior to 1850, however, the most commonly used method for securing air propulsion was the chimney draft. Tall towers with fire grates at the bottom were built specifically for the purpose of creating a circulation of air. A later development was the placing of hot water or steam pipes in the chimney or duct, to accelerate the draft.

Ventilation has always been closely associated with heating. Hot water, and particularly steam heating, paved the way for the development of indirect or hot-blast heating, which is so common to-day.

At the birth of the American Republic, rooms were warmed with grates and stoves. The heating and ventilating equipment of Independence Hall in 1776 consisted of two open stoves (which cost about $135) and wooden shutters or ventilators. Hot-water heating was being tried out for chicken incubators in France and England. James Watt was warming his study with an iron box connected to a boiler in the room below. The difficulties of this period had to do with the control of heating.

By 1850 steam had begun to find extensive use, and this opened the way for the rapid heating of air in quantity, in other words, indirect heating. This method, also known as the plenum, or the hot-blast, system grew in favor and, in 1867, Chicago installed its first mechanical plant with indirect heating, for a school building. From a main duct in the basement two tin ducts were carried up through the wardrobes, terminating in openings in the classroom about 12 in. above the floor. Deflectors over the openings directed the incoming air down toward the floor. Needless to say, there was considerable complaint of drafts.

Between 1850 and 1900 mechanical ventilation found greatly extended use, but there were no radical changes in methods or principles. In fact, in looking back over the history of the subject, it is rather striking that no particular method characterizes a particular period. In 1870 discussion waged over the relative merits of the hot blast and the split system. The same dis-

cussion is rife to-day. The Tobin tube and the Sherringham valve, of the early Nineteenth Century, were methods of introducing air directly through the wall of a room. This method is exemplified to-day in the window deflector and the unit ventilator, to which the fan has been added.

The problem of ventilation has been how to warm an inclosure in cold weather, or cool it in warm weather, and at the same time circulate air in such a manner as to remove odor, conserve heat or cold, and not cause unpleasant drafts. The matters at issue have been whether to heat directly or indirectly, whether to admit air at the top of the room or the bottom, whether there should be a single air inlet to the room or many, whether air should be forced through an inclosure or exhausted, or both.

The improvements that have taken place, in addition to artisanship, are largely refinements in control and increases in capacity, which have been made possible through developments in the use of steam and electricity.

Air Conditioning

Since 1900 the most notable changes have been in the field of air conditioning, humidification, and dehumidification. A form of humidification was practiced in 1838 when the air supplied to the British House of Commons was made to pass through a sheet of water dropping from perforated pipes. The development of humidification, however, took place mainly in the textile industries, where high moisture content was necessary to prevent the breaking of threads. Humidification by the release of steam from a jet gave way to the use of a pan of water heated by steam pipes, and known as an evaporating pan. This, in turn, was followed by the combined air washer and humidifier, consisting of spray nozzles, the water being circulated by a rotary pump. The degree of humidification was controlled by a thermostat in the wash-water or by a humidostat in the air duct.

Effect of Changing Theories on Recent Practices

In the last twenty-five years changing theories of ventilation have left their mark on progress. So long as respired air was supposed to be poisonous, there could be no let up in the amount of air change. This viewpoint led to unnecessary restrictions, and actually retarded progress. An illustration is the English weaving sheds where an upper carbon dioxide limit of 6 parts per 10 000 was specified. This did not bring comfort, for the places reeked with steam and were unbearably hot. After a thorough study of the situation —and even at the suggestion of the operatives—the air flow was cut down, the CO_2 limit was raised to 11 parts and, what was most important to comfort, an upper limit was placed on the wet bulb temperature.

Another instance is in mines, where an unnecessarily high standard of oxygen was specified. Later, it was learned that lower concentrations of oxygen were preferable as this reduced the danger of mine explosions.

For years a controversy has waged over the ventilation of school buildings, fresh-air enthusiasts speaking of mechanical ventilation as "canned air". Engineer extremists, on the other hand, would not even apply the term ventilation unless propulsion of air was by mechanical means. Once having

recognized the non-poisonous character of respired air, there should not be the slightest objection to window ventilation and gravity exhaust, although this method will not insure a continuous air flow equivalent to the 30-cu. ft. standard. For this reason, laws which require mechanical ventilation in school buildings, indirectly through the provision calling for 30 cu. ft. of air per min. per person, are unnecessarily severe, and should be amended.

A clearer understanding of the ventilation elements made possible also the practice of recirculating air, thus saving fuel cost. This would have been deemed impossible at a time when a different attitude prevailed with reference to carbon dioxide and organic matter. The Young Men's Christian Association Training School at Springfield, Mass., found this procedure entirely practical and economical, during the course of experiments in 1913. The method has now found application in theatres and factories, and to some extent in schools. The use of ozone to cut down odors has been tried in conjunction with recirculation, and has met with favor, particularly in the schools of St. Louis, Mo.

The unit ventilator, or combined blower, humidifier, and heater for individual rooms, has been a recent development. The apparatus is more costly, but it permits individual room control. This saves in the operation of a central plant when only one or two rooms need to be ventilated.

Ventilation Laws

Compulsory ventilating laws for public buildings and factories have been enacted largely since 1900. Regulations are to be found at an earlier period; Detroit, Mich., for instance, prescribed ventilation appliances for places of employment in 1881. State-wide acts were passed in New Jersey in 1903, and in Ohio and New York in 1905. In approximately twenty-four States there are now laws or regulations prescribing ventilation in schools and other places. For the most part, these laws require forced ventilation, with an allowance of 30 cu. ft. per min. per person. They also specify floor space and cubic space standards per occupant. To a lesser degree, temperature standards are also laid down.

Contributions of Ventilating Practice to Human Welfare

To-day ventilating engineering represents a distinct profession. Evidence of this is seen on all sides. Habitations are much more comfortable as a result, and the hazards of indoor life are unquestionably reduced thereby. The dust which used to becloud the worker in the polishing room is now removed locally. Steam vapors from vats, which formerly rendered their attendants invisible a few feet distant, may now be carried away through exhaust hoods. Basement and sub-basement space, which formerly had limited use, is utilized by cafeterias which, through the aid of electric lights, ventilating ducts, and a dehumidifying system, are made attractive and comfortable for thousands of people daily.

Ventilation, an ephemeral thing at the dawn of the century, now invades the theatre bill posters, where glaring letters give almost as much prominence

to the cooled, washed air, as to the names of the play and the comments of the critic.

The American Society of Heating and Ventilating Engineers, organized in 1895, has grown to large proportions, and now conducts a research laboratory, where the problems confronting the profession are being studied.

In spite of the advances made and the numerous illustrations of good ventilation, there is room for a much wider use of good equipment in such places as theatres and meeting halls. It is the rule and not the exception to leave a theatre after a 2 or 3-hour performance, hot, sticky, and thoroughly uncomfortable. Engineers have yet to learn how to counteract the heat production of a hall full of people so that temperatures at the close will be nearer 70 than 80 degrees.

Outlooks for Ventilation Progress

The changes in ventilation perceived in the immediate future are likely to include:

(*a*) Refinements in the control of temperature;
(*b*) The introduction of cyclic or variable air motion, as opposed to unvarying and monotonous air flow;
(*c*) The development of methods of cooling during warm weather;
(*d*) A more economical use of fuel through the medium of recirculation;
(*e*) Rewriting and modernizing legislative requirements; and
(*f*) More intensive study of plant operation.

From the wealth of evidence available, overheating is more threatening to human comfort, health, and efficiency than any other single ventilation element. There is need for continued improvement in thermostatic control and regulating instruments.

The oscillating wall fan is an improvement over the stationary type. It cools but does not chill. Intermittent air propulsion by blowers will soon be accomplished. One of the most justifiable criticisms against mechanical ventilation is its deadly uniformity. This is not an idle criticism. Outdoors one is used to the intermittent play of air on the skin. This is missing indoors, unless the windows are open. This is one of the attractive features of window ventilation, and is one of the reasons for the insistance on open windows. There is no reason why this condition cannot be quite successfully imitated mechanically. It is not the alternate starting and stopping that is wanted, but the actual reproduction of the irregular outdoor velocity cycle.

When this comes it will be possible to introduce air at lower temperatures, which will be more stimulating and fairer to the nose and throat. The intermittency of the action will produce the sensation of a breeze, but not that of a draft. It is this refreshing, draftless breeze which is more needed than anything to vitalize mechanical ventilation.

People are accustomed to think of ventilation as a winter problem, but are beginning to see its uses as a protection against the debilitating heat of summer. The circulating fan is growing in popularity. The cooled and dehumidified theatre is here. Who knows but what the gas compressor and expansion coils may cool the living and work rooms as well as the ice box?

The recirculation of air will find wider and wider application, for this means fuel economy. Large spaces with relatively slight crowding are particularly suitable for this practice.

There are numerous examples of well-intentioned, but illy worded and unwarranted legislation bearing on ventilation. This distracts attention from the really important questions, provides a ground for controversy, and hinders progress. This situation is bound to be remedied.

The literature of the subject contains many more accounts of projected works than of impartial critical tests of operating results. In order to learn the most from experience, there must be careful observations of equipment in use. This is necessary in order to verify or disprove the practical success of engineering ideas which are constantly being introduced. It is to be hoped that the research staff of the American Society of Heating and Ventilating Engineers may find opportunity to enter this field and bring forth scientific data.

Most to be desired is air that is cool rather than warm, dry rather than damp, diverse in temperature in different parts and at different times rather than uniform and monotonous, moving rather than still, and, finally, free from annoying odors and floating matter. This is fresh air. It is the object of ventilation to reproduce this condition indoors as faithfully as possible.

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INDUSTRIAL WASTE DISPOSAL

A SYMPOSIUM*

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Note.—Written discussion on this paper will be closed in **January, 1928.**

* Presented at the meeting of the Sanitary Engineering Division, Philadelphia, Pa., October 7, 1926.

INDUSTRIAL WASTE DISPOSAL

INTRODUCTION

By W. L. Stevenson,* M. Am. Soc. C. E.

The conservation of water resources is of importance to the health and prosperity of the people of the United States. Life in modern cities is impossible without adequate public water supplies, and the protection and promotion of the public health demand that such water be pure and palatable.

Industry must have water for use in processes of manufacture, and for many industries such water must be clean. In agricultural areas clean streams are needed for watering stock, and the trend toward outdoor recreation is increasing the need for clean streams for that purpose in suitable localities.

Streams, however, are the natural drainage channels of the land and of necessity must receive the rain water which falls upon their catchment area, carrying with it substances eroded from the surface of the earth and dissolved by the ground-water from beneath the surface.

Similarly, but artificially, the streams must also receive back again the water which having been used as public supplies in towns becomes sewage, and having been used in manufacturing becomes industrial waste, and being pumped or flowing by gravity from mines, oil wells, etc., carries substances from sub-surface strata. All these are hereinafter called waste waters.

The rule of law that each riparian owner is entitled to receive water from up-stream owners "unimpaired in quality" was undoubtedly just at the time it was declared and was practicable everywhere in this country when it was first settled. It may still be applied in the undeveloped and sparsely settled parts of the country, but elsewhere modern civilization, with sewered cities, industries, and the development of underground resources, has completely changed the primitive conditions which existed in the days of the pioneer.

Therefore, if real conservation of water resources is to be attained in order to make adequate provision for all the manifold and diverse uses of streams in the developed parts of the country, it will be necessary to adopt and carry out, on a nation-wide scale, a broad comprehensive plan for the sanitary conservation and the prudent utilization of the water resources.

Such a plan should be based on the fundamental principle that conservation is primarily intended to render the water resources increasingly useful for the well-being of the Nation, but it is not intended to return them to a state of pristine purity nor arbitrarily to estop the discharge of all unpurified waste waters to them.

Such a comprehensive plan, therefore, should include:

(*a*) An order of precedent of use of water, in which the first and highest use generally will be for sources of public water supplies for domestic and municipal purposes.

(*b*) An allocation of the use of river systems, streams, or even parts thereof, for whatever purpose best serves the general public interests and is not directly incompatible with the local interests.

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(*c*) A recognition of the natural capacity of rivers to assimilate inoffensively and purify ultimately limited quantities of certain kinds of waste waters.

(*d*) An equitable and just assignment of this natural assimilating capacity among those parties who properly discharge waste waters to the rivers and also an equitable and just assignment of responsibility for purifying before use the water of the streams among other parties who divert the water for various proper purposes.

(*e*) An economical and practicable procedure to determine the assimilating capacity of streams for various kinds of industrial wastes and to find reasonable and practicable ways and means of treating industrial wastes to produce effluents capable of inoffensive assimilation.

In some States discretionary power is vested in the executive officer, board, or commission, which administers the statutes relative to sewerage, and the discharge of sewage and industrial wastes. This makes possible the adoption of a comprehensive plan such as that described. In the statutes of other States there are inflexible and positive prohibitions against any discharge of enumerated substances to streams, or against groups or classes of wastes deemed detrimental to certain uses of streams, and in such cases worth while results cannot be obtained where the prohibition is impracticable or merely idealistic. Also, anti-stream pollution statutes are generally of State-wide application and do not differentiate between the widely varying characteristics and needs of the different sections of the State.

The folly of this principle is being corrected in the matter of municipal development through the zoning of cities, whereby the variation of requirements for residential and industrial uses is recognized and defined. The principle of zoning is applicable to a comprehensive plan for the conservation of water resources.

As this is an introduction to an industrial waste symposium, only that part of the proposed comprehensive plan concerning the assimilating capacity and methods of treatment of waste waters will be considered herein.

Many scientific investigations have been made of the conditions of streams receiving sewage, so that definite knowledge is now available as to the amount of sewage which streams can inoffensively assimilate. Also, processes have been developed and established whereby sewage can be treated to any required degree. As regards sewage, therefore, the needed data are now at hand, and the problem is largely one of determining the required degree of treatment of the sewage, if any, the designing of structures, providing funds, constructing works, and then efficiently operating them.

Such, however, is not the case with industrial wastes. For many years, both technical and lay investigations have been made concerning the treatment of various industrial wastes. All over the country industrial waste treatment works have been installed—some entirely satisfactory and efficient, others, regardless of the expenditure of considerable funds, failures or gravely inefficient.

Ofttimes only the engineer who designed the works and the company that operates them know any of the details of either the successes or the failures. Such data are not generally available to the Engineering Profession, nor to industry as a whole.

It is certainly uneconomical for any one company engaged in a certain industry to expend its own funds in research and experimental installation in order to find out how to treat an industrial waste which is or may be a common problem to many other companies engaged in the same industry, except in special cases or for the profitable recovery of by-products.

For the solution of such problems in any State a more practicable and economical procedure is for the State agency which administers the statutes having to do with discharge of industrial wastes to enter into co-operative agreements with the various groups of industries, whereby funds are provided to be expended by a committee of competent technical experts in investigations to determine:

(1) The polluting strength and characteristics of the ingredients of the industrial waste.

(2) The assimilating capacity of streams for the industrial waste, untreated and treated to various degrees.

(3) Reasonable and practicable processes of treatment to provide a progressively increasing quality of effluent so as to meet the varying requirements of different streams.

(4) Cost data of construction and operation.

There are technical problems which are common to several industries and questions to be answered of common interest to different industries, such as the basic principles underlying standards of cleanliness of streams used for various purposes. To solve these technical problems and answer these questions there should be co-operation between the industrial waste committees of the various States.

Certain industries operate in many States and in such cases economy of funds and enlargement of scope of work may be attained by providing for coordinating or even unifying investigations of industrial waste disposal through a National committee made up of representatives of the State officials, the technical experts, and the executives of the industry in those States where such co-operative agreements may be put into effect. Such a project is not a theory. It has been inaugurated.

In Pennsylvania, technical committees have been created and are at work on the unsolved problems of disposal of tannery wastes and on pulp and paper mill wastes.

In Ohio, committees have been created for the study of the waste disposal problems confronting the canning, dairy, acid, iron, and paper industries.

In Wisconsin, pea-canning and pulp and paper committees are in existence and, in Michigan, a tannery committee has been organized.

The American Paper and Pulp Association has created the "National Stream Purification Committee for the Pulp and Paper Industry", consisting of representatives of the industry and of five State officials.

Therefore, it appears possible to build on the existing foundations a superstructure which will be a sane, practical, comprehensive plan for the sanitary conservation and prudent utilization of the water resources of the United States.

TREATMENT OF WASTES FROM OIL REFINERIES

By Robert Spurr Weston,* M. Am. Soc. C. E.

The treatment of wastes from oil refineries is different from that of most industrial waste liquids. This is because oil-refining processes differ so greatly from those the wastes of which the engineer is usually called upon to treat.

An outline of typical refinery practice has been presented† by the writer and will not be repeated here. It is sufficient to state that in an ordinary refinery much water is used for cooling; various petroleum products are treated with chemicals and washed with water; and, at times, there are leakages from condensers and from the multitude of tanks and pipes containing the oil and oily products. All these waste liquids reach the refinery sewer, together with many results of bad or experimental processes and many operators' mistakes.

So much for the waste liquids. There are also waste semi-solids and solids. Refinery practice varies widely. In some refineries the crude oil is only "topped", that is, the gasoline is distilled, the residue being used for fuel. In others, the whole range of products from petroleum ether to coke, is made. The heavier products, petroleum residues, waxes, and coke, are sometimes denser than water.

As time goes on, one may expect that the waste liquids from oil refineries will become more and more complex, because the petroleum industry is still in its infancy. Thus far, oil refining has existed largely for the separation and sale of the various groups of elements in crude oil. In the future, it is certain that, in addition, as many products will be made from petroleum as are now made from coal-tar. When this day comes, one may expect more complications in the treatment of refinery wastes.

Refinery sewers are usually on the combined system, and the sewer inlets from refinery yards (seldom paved) carry much dirty storm water in addition to the usual leakage of ground-water and the refinery wastes themselves.

Most refineries have a separator into which the refinery sewer discharges. Usually this is a simple basin with skimming-boards and swinging, floating arms by which the oil is decanted from the surface of the separator and pumped back to the refinery to be passed through the process again. In many refineries the basin is divided into several compartments to facilitate the skimming of oil.

From one point of view, separators for liquid wastes alone may be considered as subsiding basins upside down. They receive oil and water and separate them by gravity. Unfortunately, the operation is not so simple as this conception. In many of the processes, and in tanks and pumps, emulsions are formed which are difficult to destroy. Also, treatment with various

* Cons. Engr. (Weston & Sampson), Boston, Mass.

† *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 383.

chemicals causes the formation of oily products with densities about the same as that of water, or greater. In many refineries, only the easily recoverable oils are skimmed off, the remainder being wasted. All these add to the difficulty of separation. After a time the oil discharged with the waste water inevitably separates, usually as a film on the surface of the body of water into which it is discharged.

The coke and heavier residues present a more difficult problem than the oils. Many of these residues are hot and fluid when discharged, but become plastic solids when mixed with the cooler wastes in sewer and separator, where they subside as pasty masses difficult to remove.

The design of the separator is usually simple, shallow basins with either vertical or sloping sides being customary. In the best practice, the wastes are passed through fore-basins where the larger part of the oil is readily skimmed off and the mass of settleable solids collected. These fore-basins greatly reduce the volume of water accidently skimmed off with the oil and minimize the labor of removing the heavier solid matter from the system of basins.

Probably because refineries have been located on large bodies of water where the discharge of waste therefrom has not been particularly objectionable, few have done more than to build simple separators. This has been done more to save oil than to protect the water receiving the effluent. A few other refineries have been compelled to go further.

In the manufacture of lubricating oils, wastes which are emulsions of oil and water are quite prevalent. As is well known, the permanence of oily emulsions is more or less dependent on chemical reaction, and although it is probably possible to separate and treat these preliminary wastes by adjusting the reaction and separating the oil by centrifugal machines, the practice is not general.

The coke residues are removed from separators either by hand or by hand and bucket hoist. There seems to be much opportunity for improvement in this regard, and apparently there is no reason why automatically cleaned tanks of the type used so successfully for sewage should not be adapted to oil refinery wastes.

Thus far, the chief factors in the purification of refinery wastes are gravity and time, and regarding the latter, there is no standardized practice. There are large refineries with small separators and *vice versa,* and it is rarely ever known what periods of detention are necessary or desirable.

The separator of one refinery was observed by the writer's firm for a year. The brackish water (30 to 50% sea water) used for condensing was returned through the sewer and separator, with its content of oily matter. The separator had a capacity of about 4 hours' flow, and the effluent from the basin generally contained from 5 to 19 parts per million of oily matter (ether soluble). There were, however, two days on which the content of oil rose to 42 and 78 parts per million, respectively. The variations are shown on Fig. 1. The average quantity of oil found was 12 parts per million. The body of water into which this effluent discharged had an average oil content of 3.1 parts per million. This refinery was in a large city, and the

presence of oil in the water receiving the effluent was due only in part to the discharges from the refinery and in part to the discharges from ships and from the sewers of the city, where much oil was used for fuel.

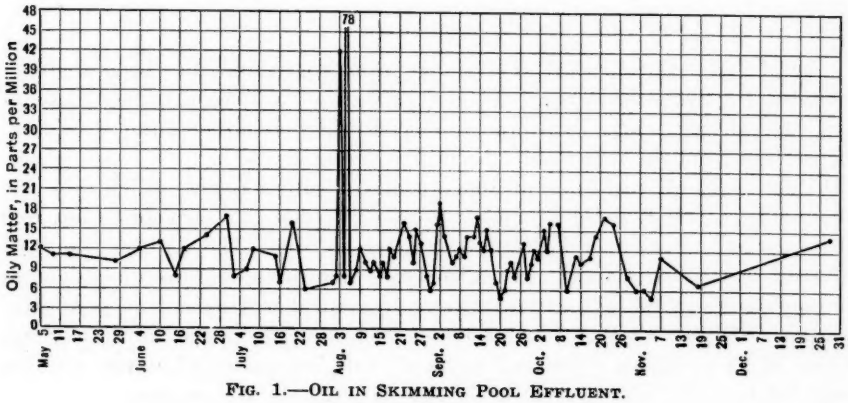

FIG. 1.—OIL IN SKIMMING POOL EFFLUENT.

Two other separators that were tested, produced effluents having the characteristics given in Table 1.

TABLE 1.

	PARTS PER MILLION.	
	Separator *A*.	Separator *B*.
Total solids	17 540	17 200
Loss on ignition	3 850	3 815
Turbidity	95	300
Oils (ether soluble)	40	40
Acidity, as H_2SO_4	.....	421
Alkalinity, as $CaCO_3$	110	.....

The conclusion from Table 1 is that at present the art of treating oil-refinery waste is rather rudimentary, and although methods are effective to a certain degree there are many improvements which suggest themselves.

A satisfactory treatment works should discharge an effluent practically free from settleable solids and one containing less than 20 parts per million of oily matters (ether soluble).

DISPOSAL OF DRAINAGE FROM COAL MINES

By Andrew B. Crichton,* Esq.

In the past the disposal of drainage from coal mines has been a simple and inexpensive proposition. As coal properties have been opened and the workings driven, water is always developed, which is either pumped or flows by gravity from the mines into the nearest waterway.

Practically all coal-mine drainage contains sulfuric acid, which soon impregnates the fresh-water streams into which it flows to such an extent that they are unfit for either industrial or domestic use. The natural alkalinity of fresh-water streams will neutralize large quantities of mine water, but as it requires 80 to 100 gal. of fresh water to neutralize 1 gal. of average mine water, it will be seen that dilution is not long effective, in the mining regions. Some of the principal rivers and many of their important tributaries are now acid for long distances during their flow through the coal regions.

The stream-pollution problem from mine drainage is by no means new, and in the mining regions has long been the cause of great inconvenience and expense both to the mining companies and the public. Until quite recently it has been strictly a mining community problem. As the mining companies usually controlled the communities, if a water supply were destroyed by mine drainage no attempt to treat the water was made if other unpolluted supplies could be found. In many instances large sums have been spent in building treatment plants, which because of unsatisfactory results have later been abandoned and unpolluted water brought long distances at great expense.

The largest industries and public utilities have suffered most from stream pollution from mine drainage, because of their large and growing needs for pure water. Nearly one hundred water companies in Pennsylvania have similar problems, which are daily becoming more serious.

For years the mining industry has been considered paramount, but one is forced to admit that although coal is necessary, water is necessary for human existence; that one is dependent on the other, and that both are vital to the welfare and prosperity of the Nation. The development of the coal industry has gradually progressed during most of the past century, but has not nearly reached its maximum. Whatever ills have come with it have likewise been long in developing, so that good judgment and patient investigation, rather than drastic legislation, should govern in handling this problem.

Importance of the Coal Industry

The Nation could not long survive without the coal industry. It is the principal source of power and serves mankind in so many and varied capacities that one can hardly recount its blessings. The value of its gas and chemical products to civilization has never been properly appraised or appreciated. With the decline of oil resources the country will become even more dependent

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on the coal industry, and nothing should be done which will needlessly add to the cost of producing coal, or that will in any way affect its usefulness to mankind.

Coal is produced in twenty-eight States and is fairly well distributed. The capacity of the developed mines is said to be 750 000 000 tons annually. The United States produces about one-half the world's supply of coal and the value of this annual product is more than $1 500 000 000 at the mines. Including freight charges and other transportation costs, the ultimate cost to the consumers would be more than $4 500 000 000. Proposed National legislation prohibiting the discharge of acid mine drainage into the streams might easily add $200 000 000 to the annual coal bill, with doubtful results or without materially improving the fresh-water supplies.

Additional Legislation not Advisable

Until some satisfactory method has been found to treat or take care of mine drainage no additional legislation is needed. It has been said by many that this is a National issue, that any legislation requiring the disposal or treatment of mine drainage that will add to the cost of producing coal, should bear equally upon the whole industry. The operators of one State or district should not have any advantage over another as a result of legislation, but National legislation will not bear equally upon all districts, because the problem is not the same everywhere.

The problem is most acute in Western Pennsylvania, Northern West Virginia, and Ohio, and under any plan the cost of disposal will be greater in those districts than elsewhere. Mine drainage from Northern West Virginia flows through Pennsylvania, so that Pennsylvania legislation without any control of West Virginia mine drainage would be ineffective and would add an additional burden to the coal operators of Pennsylvania. Any apparent disregard of the rights of others or the protection of water supplies by the mining industry may in a measure be attributed to the principle laid down by the Pennsylvania Supreme Court in the now famous Sanderson case.

Court Decisions Affecting Mining

In 1886 Sanderson bought a property in the City of Scranton through which flowed Meadow Brook, a pure, unpolluted stream. He built a dam and developed a water supply for his own use. About the same time the Pennsylvania Coal Company opened a coal mine which soon produced acid mine drainage, destroying the use of the brook water. Sanderson brought suit for damages resulting from loss of the stream. The case was twice tried in the Courts of Lackawanna County, Pennsylvania, and was twice before the Supreme Court of Pennsylvania. The first Supreme Court decision affirmed the lower Court's award of damages to Sanderson; Justice Paxson filed a strong dissenting opinion, which was sustained in the second Supreme Court decision.

The Court took the position that if Sanderson could collect damages, every riparian owner thus affected could do likewise, and if they could collect damages they could also enjoin the pollution of streams by mine

drainage, which would practically stop all mining operations, except by consent of the lower riparian owners; that trifling inconvenience to particular persons must sometimes give way to the necessities of a great community, especially where the leading industrial interest of the State is involved. The Court further stated in its opinion that the Coal Company was making the natural and ordinary use of its property, and that Sanderson, with others, was then securing an abundant supply of pure water from other sources, but that it would not say that a case "may not arise in which a stream, from such pollution, may not become a nuisance, and that the public interests as involved in the general health and well being of the community may not require the abatement of that nuisance".

Pollution a Problem of Long Standing

For the past forty years one water supply after another has been abandoned in Pennsylvania, due to mine drainage pollution, until now there are few available supplies not polluted or subject to pollution. The protection of water supplies, to meet the constantly increasing demands for domestic and industrial needs, is, therefore, imperative.

In 1905 and shortly prior thereto the Pennsylvania Railroad Company was put to considerable trouble and expense by a water shortage. Old sources of supply had become polluted, and attempted treatment was unsatisfactory, resulting in engine failures and serious delays in operation. Millions of dollars were appropriated to secure an adequate supply for the Company's present and future needs. It was then not possible to secure supplies reasonably convenient to points of consumption which were not being polluted nor subject to pollution from mine drainage.

Development of Indian Creek Supply

The Mountain Water Supply Company was organized in 1905 and appropriated the waters of Indian Creek to supply the Pennsylvania Railroad System in Southwestern Pennsylvania, as far west as Pittsburgh. A large storage dam was built about 4 miles from the mouth of Indian Creek. The drainage area above this point is 110 sq. miles, of which 55 sq. miles are underlaid with the lower productive coal measures. At the time there were numerous small "country bank" openings for local supply for the farmers in the valley, but no commercial development of the coal lands. Several years later the Indian Creek Valley Railroad Company constructed a standard gauge track from Indian Creek, a point on the Baltimore and Ohio Railroad, up the valley to its headquarters, following which development of the lumber and coal resources was begun, but there was no material development until about 1917.

In addition to supplying the Railroad Company, the Water Company furnished water to several municipalities in Western Pennsylvania, supplying about 75 000 people. Active coal development began on a large scale, and it was apparent that this important water supply would be quickly destroyed for the purpose intended, unless something were done to keep the acid mine drainage out of the stream above the dam. During this time both the coal and water interests conducted studies and experiments in an effort to find

some satisfactory method of treating acid mine drainage that would permit the mining of the coal without the destruction of the water supply, but without success. The Water Company finally appealed to the Courts for an injunction to restrain the coal companies from discharging acid mine drainage into Indian Creek above the dam.

Indian Creek Pollution Suit

This case is known as the "Indian Creek Pollution Suit". The Fayette County Court decided there was no public use of the water and that preventing the mining companies from discharging water into Indian Creek would deprive them of the use of their property. The Court refused to grant an injunction restraining the mining companies from discharging mine water into this stream. The Pennsylvania Supreme Court reversed the lower Court, declaring that it was not a question of property rights, but that it was a nuisance to pollute the stream, and that the mining companies should not, after a certain period, discharge the mine water into Indian Creek or its tributaries above the dam of the Water Company. This opinion, which was concurred in by the United States Supreme Court, states:

"It is controlled by one fact and a single equitable principle: the fact that the stream has been polluted, and the principle that this creates an enjoinable nuisance, if the public uses the water".

The difference between the Sanderson and the Indian Creek decisions is that Sanderson, an individual who had access to another good supply of water, was not permitted to stand in the way of Pennsylvania's greatest industry, whereas in the Indian Creek case the public, also represented by the Commonwealth, is fighting to preserve one of the last available pure water supplies of the State, and the present decision merely carries out one of the principles enunciated in the Sanderson decision. As a result, there has been considerable discussion as to what constitutes mine drainage, or mine water. The water companies sought an injunction to restrain the mining companies from discharging acid mine waters into Indian Creek. It was admitted by both sides that eventually this mine drainage would destroy this water supply for the purpose for which it was then and is now being used. Some of the leading chemists, water-works, and mining experts of the United States were witnesses in the case, and there was no disagreement as to the fact that continued pollution by mine drainage would eventually utterly destroy this stream and render it unfit for any use.

There was much testimony introduced regarding the treatment or disposal of mine water, without material disagreement regarding it. Nearly every one in discussing "treatment" refers to neutralization by the introduction of lime. A prominent chemist, in a recent paper prepared especially for the purpose of fixing accurate and understandable definitions, repeatedly mentions "purifying" mine water by introducing lime. There is much confusion as to what is meant by treatment. The Conservation League wants a water that will not kill fish or vegetation; the Army engineers apparently want an alkaline water that will not affect the metal parts on the locks and dams,

or interfere with shipping; the water companies, for the domestic and industrial user, want a water fit for human consumption and industrial use; and every one would like the iron oxide kept out of the stream beds.

Treatment of Mine Water

The neutralization of acid mine water by the introduction of lime does not make the water fit for either domestic or industrial use. This matter was clearly before the Court, and was undoubtedly the reason for the decree that the mine waters must be kept out of the stream. The meaning of that decree is surely that the water supply is to be preserved in condition for use, and the only safe way to do that is to keep the mine water out of the stream.

Of the total rainfall, it is estimated that only 25% penetrates into the ground, and only a small proportion of these percolating waters reach great depth. The sandstones and shales overlying the coal seams become great reservoirs, storing vast quantities of water. Immediately under the coal seam is usually found a stratum of impervious fire-clay through which these percolating waters do not penetrate. It is at these levels that surface springs are found, and through which the ground-waters find their way back into the streams. In the coal regions, the surface springs are an important guide in prospecting for coal. Prior to coal mining, these percolating waters, although penetrating the coal seams and carrying varying amounts of dissolved solids, do not become acid.

It requires the combined action of air and water to dissolve the iron pyrites, found in the coal seams, to form sulfuric acid. This acid combines with other elements in the water, usually calcium, magnesium, and iron, and forms sulfates, so that waters not showing free acid, but containing magnesium and calcium sulfates, are often highly corrosive and are hard and unfit for either domestic or industrial use. There is quite general agreement that when a water supply contains 4 grains of sulfates per U. S. gallon, it requires softening. When the pollution exceeds 12 grains, treatment is no longer effective because the large quantity of soda ash required so heavily charges the waters with solids as to cause foaming in boilers and renders it undesirable for domestic use. It will be seen, therefore, that if the waters require treatment with the small percentage of coal mined, with continued mining it will be only a comparatively short time until treatment by any known method is no longer effective.

On exposure to air the iron constituents become less soluble and are precipitated, causing the yellow deposit in the beds of the streams so familiar to every one in the mining regions. After the rocks are coated with this iron deposit, the acid in the stream has no further access to whatever neutralizing elements the rocks may contain, and the only further change in the acid content of the stream waters is from the natural alkalinity of fresh-water tributaries, the process of dilution.

In the course of mining, as the coal is removed, the inferior coal, carbonaceous shale, iron pyrites, and other refuse material, as far as possible, is stored in the mine. As mining proceeds, water is developed from the coal and the

overlying strata and flows promiscuously through the mines, coming in contact with this refuse material containing the acid-forming elements, and reaches the surface as acid mine drainage. In upward of 300 mines examined, only 4 or 5 have been found in which the drainage was not acid. In one mine it was due to dilution from surface waters encountered in shaft-sinking, and, in others, to the percolating waters coming in contact with limestone. Large quantities of this refuse material are also hauled from the mines and stored in great piles around the mine openings, and there is less chance of control of the acid from these dumps than that from the mines. Any contemplated plans for disposal or the treatment of mine drainage seem to have overlooked this prolific source of pollution.

Quantity of Mine Drainage

To determine the quantity of mine water to be expected from the mining of coal, 250 to 300 mines have been examined. It was realized that probably no two mines in any region are operating under exactly similar conditions, but average conditions could be expected in entire fields of about the same size operating the same measures. It was realized that the daily or annual production of coal has slight, if any, relation to the quantity of water to be expected, although that method has been so often referred to in the anthracite region, and in recent Government reports. The area of coal exhausted was considered to be the most reliable basis for an estimate, although it, too, has many factors affecting the results. Some of these factors are the thickness and nature of the overlying strata; the thickness of the seams and methods of mining, particularly regarding the drawing of pillars—whether pillars are drawn near the crop or in extremely shallow cover; the protection given to streams; and the pitch of the measures.

An acre of coal exhausted in the bottom of a coal basin might affect a large surface-drainage area. This accounts for unusually high yields per acre in some small mines. In fairly deep mining, say, 250 ft. or more, the maximum yield of mine drainage should not be expected to exceed 25% of the rainfall, the estimated penetration. However, with the drawing of pillars, serious surface breaks occur where the coal has as much as 600 ft. of cover. In such cases undoubtedly a much larger percentage of the rainfall reaches the mines, reducing proportionally the surface run-off. This increased quantity of mine drainage during wet-weather periods does not reduce the acid content of mine waters, but the waters reaching higher levels in the mine come in contact with new sources of pollution and usually contain more acid.

Effect of Mine Development on Yield of Drainage Area

In the mines examined, the area of coal exhausted and the mine-drainage flows were measured, showing in one extreme a yield of 10 000 gal. of mine water per acre per day, although the average of all mines was about 1 100 gal. per acre per day. Average mining conditions show yields of 2 000 to 2 500 gal. per acre per day where less than 100 acres are exhausted. With increased mining the rate per acre naturally declines, until an average of about 1 000 gal. per acre per day is reached.

Under average conditions the flow of mine drainage is relatively constant, and does not vary directly with rainfall. In a coal field in Central Pennsylvania where 4 300 acres of coal were exhausted, and where the cover on the coal averaged 250 to 300 ft., records of rainfall and mine drainage were kept over a period of a year. The average rainfall was 3 300 gal. per acre per day, whereas the average mine drainage was about 900 gal. per acre per day, or 27% of precipitation. The surface drainage area was much larger than the worked-out coal area, but the coal area was compact, and these figures fairly represent the percentage of percolation. The difference between the average and the minimum mine flows was only 17 per cent. Another coal operation in Pennsylvania (average cover on coal about 300 ft.; area exhausted, 2 200 acres; over 60% of which pillars have been drawn) the average rainfall was 3 000 gal. per acre per day and the mine drainage was 1 300 gal. per acre per day, showing a percolation of 43 per cent. This field is on the east side of a coal basin where additional territory than that actually mined is drained. The difference between the average and minimum monthly flow was about 14 per cent.

A drainage area of 36 sq. miles practically all underlaid with coal, of which one-half is mined, on a certain day in October, 1920, yielded 23 600 000 gal. of water, of which 10 800 000 gal., or 46%, was mine drainage. The yield of the drainage area was 1 025 gal., and of the mines, 937 gal. per acre per day. A drainage area of 62 sq. miles, 52 sq. miles containing coal, of which 12 sq. miles are exhausted, yields 41 200 000 gal., of which 7 600 000 gal. are mine drainage. This shows a yield of 1 039 gal. per acre in the total drainage area, whereas in that part of the territory developed by mining, the mine drainage yield was 989 gal. per acre exhausted per day.

If the exhaustion of less than 10% of the total coal reserves has already produced mine drainage sufficient to destroy such a large part of the water supply resources, it is apparent that continued coal mining will soon destroy the few remaining fresh-water supplies. During the summer of 1926, in the Pittsburgh District, the railroad companies were compelled to haul water for their own use and for the use of coal operations on their lines.

Quality of Mine Water

The quality of mine drainage is quite as important as the quantity. Table 2 gives typical analyses of mine waters, in grains per U. S. gallon, by Mr. E. C. Trax.

Some Pittsburgh seam mines in the Greensburg District of Pennsylvania show total apparent acidity, in grains per gallon, as follows:

Mine.	Grains per Gallon.
Keystone Shaft	98.42
Sewickley	61.74
Greensburg Nos. 2 and 3	121.68
Seaboard Shaft	137.68
Claridge	169.40
Salem	91.20
Crow's Nest	123.96

TABLE 2.—Typical Analyses of Mine Waters, in Grains per U. S. Gallon.

	Miller Shaft.	Puritan Shaft.	Sonman Shaft.	Berwind No. 35.	Yellow Run Shaft.	Morrellville.	Argyle.	Blair.	Howard.	Rogers.	Oneida.
Total solids	266.01	330.18	276.50	146.42	135.33	608.82	135.88	266.17	218.54	175.53	329.48
Probable incrustants	190.15	247.59	200.67	96.26	75.84	368.13	70.59	85.85	.....	.....	.....
Suspended matter	7.29	9.92	3.21	10.50	8.75	1.20	4.14	.....	.....	.....	.....
Iron oxide	21.56	40.10	29.37	9.22	10.10	128.11	10.27	.....	27.32	28.88	55.19
Aluminum oxide	12.90	6.73	9.14	3.38	4.38	38.21	3.73	.....	5.43	11.45	8.72
Calcium oxide	30.55	27.17	20.90	16.47	9.60	20.76	13.13	7.14	29.47	11.40	27.62
Magnesium oxide	9.05	16.25	12.01	9.77	5.66	15.63	4.79	5.57	12.73	5.49	15.33
Sulfuric anhydride (SO_3)	129.40	157.52	130.84	72.74	57.50	295.64	70.93	141.89	85.79	55.17	111.95
Sulfuric anhydride (acid sulfate)	20.82	37.33	37.68	10.04	11.67	87.06	21.88	16.47	.....	.....	.....
Free sulfuric acid (H_2SO_4)	34.42	61.25	45.50	23.62	28.00	247.06	26.47	115.00	25.04	41.53	50.31
Chlorine	0.70	0.82	0.58	0.47	0.76	2.27	2.70	0.29	.....	.....	.....

The average total acidity at sixty mines of the H. C. Frick Coke Company was 100 grains per U. S. gal. Analyses of the waters of some of the polluted streams in Pennsylvania, at much above minimum flows, are given in Table 3.

TABLE 3.—Analyses of Waters of Polluted Streams in Pennsylvania.

Name of stream.	Grains per Gallon.		
	Sulfuric anhydride, SO_3.	Sulfuric anhydride acid sulfate.	Free sulfuric acid, H_2SO_4.
West Branch, Susquehanna River, at Moss Creek	44.40	5.90	10.60
Quemahoning River, at dam	4.20	0.40	0.50
Conemaugh River, at Portage	35.23	6.80	12.40
Conemaugh River, at South Fork	24.25	5.40	9.70
Shade Creek, at mouth	4.72	1.00	1.40
Paint Creek, at mouth	37.70	7.40	14.10
Red Stone Creek, at mouth	32.89	6.77	9.33
Jacobs Creek, at mouth	19.49	3.83	8.03
Sewickley Creek, at Hunker, Pa.	40.27	9.80	16.33
Turtle Creek, at Wilmerding, Pa.	48.40	16.33	8.75

Neutralization

In treating acid mine water the custom is to neutralize it with lime, converting it into hard water, and then to add soda ash to soften it. Most waters that have been treated with any degree of success have contained a very low percentage of sulfates, as compared with mine drainage. The writer does not know of any successful attempt to treat mine drainage of the quality encountered in the Pennsylvania coal fields. After the decree of the Court in the Indian Creek case was made effective, three of the mining companies built plants to neutralize the mine water by the introduction of hydrated lime, as a compliance with the decree, but this attempted method is very expensive and is not satisfactory.

In Table 4 is given the analyses of mine water before and after treatment with hydrated lime at the plant of the Melcroft Coal Company, made for that Company by Mr. J. O. Handy, Chemist.

TABLE 4.—Analyses of Mine Water Before and After Treatment with Hydrated Lime.

(Acidity and Alkalinity Expressed in Parts per Million as $CaCO_3$.)

	Raw.	Treated.
Total acidity	940	.....
Alkalinity	.....	5
Sulfate (SO_4)	1 600	1 500
Iron (Fe)	250	2
Aluminum	5	1
Calcium	200	630
Magnesium	78	43
Silica	40	8

From Table 4 it will be apparent that there is little change in the total sulfates before and after treatment, and that the water is still unfit for use. Besides, the Melcroft water is not what would be considered a bad mine water, or nearly as high in acidity as the average mine drainage of Western Pennsylvania. The companies are now building a tunnel to carry the mine drainage to a point below the dam before discharging it into the stream. This may be a compliance with the Court's decree in the Indian Creek case, but it does not solve the problem—the disposal of mine drainage.

The cost of neutralizing mine water has been variously estimated by different chemists at 15 to 25 cents per 1 000 gal. This depends on the cost of lime, the cost of gathering the water to one central plant, the capacity of plant, the labor, and the handling of the sludge which alone is an exceedingly difficult and expensive, if not impossible, proposition. The cost of constructing a plant for lime treatment would be about $100 per 1 000 gal. daily capacity, thus making a 1 000 000-gal. plant cost $100 000.

Quantity of Drainage and Cost

A recent estimate of the acreage worked out in the bituminous mines of Pennsylvania, based on the total tonnage exhausted, would indicate a total production of mine drainage of 750 000 000 gal. daily. The cost of lime treatment plants for this enormous quantity of water would be $75 000 000. The annual cost of treatment, at 15 cents per 1 000 gal., would be $41 062 500, and at 25 cents per 1 000 gal., $68 437 500. It is estimated that the mine drainage reaching the streams from anthracite mines is 700 000 000 gal., daily, so that these estimates for the entire State of Pennsylvania would be about doubled.

It must be understood the cost mentioned is for neutralization only, and that even after this treatment, which is the one most often referred to, the water is still unfit for either domestic or industrial use. To soften the water would cost more than double the estimate for neutralization, and even with that the value of the product, or the final result, would be doubtful. It will be seen that, with an average annual production of 160 000 000 tons in the bituminous fields of Pennsylvania, the cost will be from 26 to 43 cents per ton for neutralization, and from 50 to 90 cents per ton for softening.

Difficulty of Handling Sludge

In the bituminous fields of Pennsylvania alone there would be more than 4 000 tons daily, or 1 460 000 tons yearly, of sludge to handle. Even if dried and piled outside, it would be likely to reach the stream beds with every rain storm, and would eventually fill them. This iron oxide may be used for purifying gas and in the manufacture of paint, but it has a very limited market, and with such quantities as would then be available it would be without value. To mining men it would be nothing but slimy yellow mud, too thick to pump and too thin to shovel. In the writer's opinion, the disposal of this sludge presents almost as expensive and difficult a problem as mine drainage itself.

There have been other methods of treatment suggested, but none has been found satisfactory, from one cause or another. By the use of barium salts it has been found that calcium sulfates can be removed from water, but the cost is prohibitive, and as barium is poisonous the Department of Health would not permit its use. The electrolytic process has been suggested as a possible solution for the removel of sulfuric acid and its compounds from water, but the cost would be excessive, and the handling of sludge would still present its difficulties.

Satisfactory Treatment Yet to Be Found

As far as known, there is no satisfactory method of treatment of acid mine water in quantities such as are now being produced by the mines of the United States. Acid wastes are being treated in small quantities by lime, and as a result extravagant claims are being made as to what could be done with coal mine drainage. It is apparent that these claimants do not realize either the bad quality or the enormous quantities of mine drainage that would have to be handled. Increasing the alkalinity of the stream by the introduction of lime in the mine water would greatly increase the assimilating power of the stream, but in most cases there is so much mine water that dilution would not be effective, even after the high cost of lime treatment. With these tremendous costs in Pennsylvania alone, the cost of treatment for the coal industry throughout the Nation would be staggering, and never could be justified.

In addition to the acid drainage from coal refuse dumps, there are millions of gallons flowing daily from old abandoned mines, most of which would have to be handled in some satisfactory manner before treatment of the remainder would render results worth while. In many places the sealing of abandoned mines would be helpful, by preventing the flow of mine water; perhaps shutting off the air would prevent the formation of sulfuric acid, and thus more nearly restore original conditions, but this likewise would be difficult and expensive, and the question arises as to who is now responsible?

Legislation has been introduced in Congress which, if passed, would make it unlawful within the limits prescribed by the Secretary of War to discharge any free acid into navigable waters or their tributaries. It places in the hands of the Chief of Engineers, U. S. Army, the power to require neutralization of acid water or wastes, and provides a fine for each offense, of $500 to $2 500, or imprisonment of not less than thirty days, or more than one year.

The State and individuals have spent thousands of dollars in an attempt to solve this problem, but without success, so that the time has surely come for wise Governmental action—not the hasty passage of drastic laws as now proposed, that would so greatly embarrass the mining and manufacturing industries, but a sane public policy that will tend to preserve the water supplies at as little cost to the community as possible.

One thing seems certain, that is, that coal cannot be mined without destroying the water in the mining regions. Therefore, the only sure and practical method of conserving the water supply is to stop mining coal in the few little developed areas yet remaining. This cannot well be done as between individuals, and the Government or the State is justified in purchasing, if necessary, under some plan to be devised, coal lands necessary to the protection of the few remaining water supplies in Western Pennsylvania and elsewhere. That the State of Pennsylvania recognizes the importance of this problem is evidenced by its participation with the water companies in the Indian Creek case.

The leaders in the coal industry are beginning to realize the seriousness of this situation, and that the water supplies must be preserved. There would be no objection to spending a few cents per ton of output if, in doing so, the problem could be satisfactorily solved. Some coal operators of Western Pennsylvania are suggesting that a supply of water be obtained from Lake Erie when mine drainage pollution becomes excessive. Further drains on Lake Erie would not likely be permitted, even if such a plan were feasible. However, this scheme is no further afield, and would not cost as much as the proposal that all mine drainage be treated. Any solution must be economically sound, and the writer is certain that it would cost less in Pennsylvania and elsewhere to preserve the present unpolluted streams and to stop mining in some little developed areas, in order to protect the water supplies for the present and future.

The policy of the Pennsylvania Department of Health, to protect all unpolluted streams; to stop further pollution of all streams that can be restored; and to use those now destroyed for carrying sewage, industrial wastes, and mine drainage, is the first real step toward a solution of this problem. It is commendable, and undoubtedly will result in much good at the least possible cost to the community.

PULP AND PAPER MILL WASTES

By G. K. Spence,* Esq.

The first paper mill in America was established in 1690 at Germantown, near Philadelphia, Pa., by William Rittenhouse. Since then the manufacture of pulp and paper has expanded until at present it constitutes one of the major industries of the United States.

Paper in various forms has come into use for so many purposes that it is now an essential necessity of domestic and industrial life, and it was so considered during the World War.

There are 844 paper mills and 310 pulp mills in the United States, located in 34 of the 48 States. New York State leads with 244 pulp and paper mills; Wisconsin is next with 115; followed by Massachusetts with 112; Pennsylvania with 87; Maine with 75; Michigan with 71; Ohio with 58; New Jersey with 44; Connecticut with 43; New Hampshire with 41; Illinois and Indiana each with 31; and so on down to four States with one mill each.

More than 8 000 000 tons of pulp, paper, and paper-board are annually made in this country, which output is valued at more than $900 000 000. The prosperity of the United States is, therefore, directly affected by the stability of the industry.

Paper is made from different pulps in various proportions, selected according to the grade of paper desired. Of the paper manufactured in the United States 90% is made from wood pulp, the remainder being made from rag, straw, jute, hemp, and other fiber-producing materials.

Rag Pulp

The rags are first sorted by hand, cut and dusted by machinery, and then passed to the cooking boilers where they are cooked under pressure with either a solution of caustic soda or caustic lime. After cooking, the liquor containing the soluble substances is washed from the pulp, requiring about 65 000 gal. of wash water per ton of rags treated. The washed pulp is then bleached with a solution of calcium hypochlorite and again washed in order to remove the last trace of bleach preparatory to using the pulp for the manufacture of paper.

Wood Pulp

There are two classes of wood pulp, namely, mechanical wood pulp and chemical wood pulp.

Mechanical Wood Pulp.—This pulp contains all the intercellular substance, the wood being ground to a pulpy mass by being pressed against a revolving stone in contact with water. This is the cheapest grade of wood pulp made, and, as the intercellular matter has not been removed, it deteriorates very

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rapidly. It is for this reason that newspapers, which are made mostly from mechanical wood pulp, cannot be preserved for any length of time.

The pulp from the grinders is run into a vat, taken up by a wire-covered revolving cylinder, and transferred to an endless felt, which, in turn, transfers it to a wooden roll on which it accumulates until the desired thickness is reached. It is then cut off and folded into laps for shipment or use in the beaters.

Chemical Wood Pulp.—There are three processes used for the manufacture of chemical wood pulp, namely, sulfite, soda, and sulfate.

The wood is prepared in practically the same manner for treatment by any one of the processes. It comes to the preparing room in 4 or 5-ft. lengths and from 6 to 10 in. in diameter. If the bark has not been removed, the wood is passed through revolving cylindrical drum barkers. By tumbling around in these drums in contact with water, the sticks by rubbing against each other are stripped of the bark which is washed through slots in the side of the drum. Some drum barkers are of the batch type; others are continuous, that is, the wood is fed in at one end and the barked sticks removed at the other. Sticks too large for the chipper are split to the desired size.

From the preparing room the sticks are passed to the chipper, which is a revolving cast-iron disk with projecting steel knives set so that the size of the chip can be regulated. The wood is fed through a slanting trough and moves forward by its own weight until it comes in contact with the projecting knives on the revolving disk. The size of chips in soda pulp mills varies from $\frac{3}{8}$ to $\frac{3}{4}$ in, whereas those in sulfite and sulfate pulp mills are from $\frac{3}{4}$ in. to $1\frac{1}{4}$ in. In chipping the wood, fine pieces, or sawdust, is produced, which is screened from the chips of the desired size.

The Sulfite Process

This process has reference to the method of manufacturing pulp through the isolation of the intercellular matter by means of bi-sulfite liquor. This liquor is made by burning sulfur and absorbing the sulfurous acid gas either in milk of lime or in towers filled with limestone over which water is sprayed. The resultant cooking liquor is calcium bi-sulfite when high calcium lime is used, or bi-sulfite of calcium and magnesium when Dolomitic lime is used, an excess of sulfurous acid being produced in each case. The intercellular matter is removed by cooking the chips in this liquor, under pressure, in large vessels termed digesters. The cooking liquor forms soluble calcium salts of ligno-sulfonic acid, and the lignin and other intercellular matter is removed from the fiber by washing and bleaching.

The Soda Process

The soda process has reference to the method of manufacturing pulp through isolation of intercellular matter by means of a solution of caustic soda. The caustic soda solution is prepared by causticizing sodium carbonate with caustic lime. The intercellular matter is removed by cooking the chips in this liquor, under pressure, in digesters similar to the sulfite digester, except that they are not lined, the lining being necessary in sulfite digesters due to

the acid action. In the soda process the cooking liquor forms soluble salts of the organic acids present in the wood, such as sodium acetate, sodium formate, sodium resinate, etc., and thus the intercellular matter is removed from the fiber by washing and bleaching.

The Sulfate Process

This process is very similar to the soda process in action, the cooking liquor being made up of a mixture of caustic soda and sodium sulfide. The sodium sulfide is prepared by smelting sodium sulfate in contact with carbonaceous matter which reduces the sulfate to sulfide. The caustic soda is prepared by causticizing sodium carbonate with caustic lime as in the soda process. The cooking takes place under pressure in digesters similar to those used in the soda process. The chemical action, although practically the same as in the soda process, is not as severe, due to the fact that there is a smaller excess of caustic soda present, and, as the cooking takes place in a reducing atmosphere, there is less tendency to over-cook or form oxycellulose. When cooking for strength, as is necessary in Kraft stock, the sulfate process is used.

In soda and sulfate pulp mills there are elaborate systems for the recovery of the soda used in cooking the wood. In both processes the stock from the digesters is washed with as little water as possible. The liquor from the stock is concentrated by passing it through multiple-effect evaporators after which the concentrated liquor is passed to rotary incinerators.

In the soda process the liquor is burned in the incinerators to a black ash containing 80% sodium carbonate and 20% carbon, which ash is later leached free from soda. The carbonate liquor is passed from the leachers to the alkali room, where it is re-causticized with fresh lime to be re-used in cooking more wood, 90% of the original soda used being recovered.

In the sulfate process the liquor is not burned as completely in the rotary, being discharged as a sticky mass into the smelters where fresh sodium sulfate is added to make up for loss in recovery. The carbonaceous matter present reduces the sulfate to sulfide, the mass being discharged from the smelter in the molten state, the excess carbon having been completely consumed in the operation.

Straw, hemp, jute, etc., are cooked in somewhat the same manner as rag stock, either caustic soda or caustic lime being used. As considerably less chemicals are used in cooking, it has not been considered economical to recover them.

Reclaimed Paper Stock

More than 2 000 000 tons of old papers and magazines are reclaimed annually. They are cut, soaked in a solution of sodium carbonate in order to free the ink, defibered, washed, and bleached. This reclaimed paper stock is mixed with other pulps in making certain grades of paper.

Paper is made by mixing the different pulps, according to the quality desired, in vessels termed beaters. At this point, china clay, talc, or some other form of filler is added to load the sheet and improve its finish; sizing material

is added to make it more water- and ink-resistant; and the proper coloring matter is added either to tint the sheet or to dye it to the desired color.

After preparation in the beater the paper stock is passed through the Jordans or refining engines where the fibers are cut to the proper size for the desired formation, and thence to the paper machine.

The paper machine consists of a moving endless wire with a lateral shake, on which the suspended stock flows, and forms into a sheet, the excess water passing through the wire. The sheet is then carried through the press rolls by means of a felt and thence over the cylindrical steam-heated dryers, where the remainder of the moisture is removed. At the end of the paper machine, the sheet is passed through highly polished steel rolls termed calanders, where the desired finish is imparted, after which it is either cut into sheets, or slit and wound on reels into rolls, of the desired size.

Pulp and paper mill wastes could, if not properly handled, enter the stream from the following sources.

1.—Bark from the wood-preparing room. This varies from 12 to 24% of the weight of the wood, depending on the kind used.

2.—Sawdust from the chipper room where the wood is reduced to the proper size for cooking. This does not amount to more than 1% of the weight of the wood chipped.

3.—Dust from preparation of the rag stock, which represents about 3% of the weight of the rags treated.

4.—Washings from the cooked stock in rag, straw, jute, and hemp pulp mills, as well as soda and sulfite wood pulp mills. These washings contain the soluble salts of the intercellular substances removed from the fibers. As the yield from rag stock is about 85% of the weight of the prepared rags, that from hemp and jute 60%, and that from straw, soda, sulfite, and sulfate pulp processes 40%, the washings will contain, respectively, 15%, 40%, and 60% of the weight of the raw materials treated, in addition to the excess chemicals used, and entrained short fibers representing about 1% of the total fiber production.

5.—Wash water from the brown stock thickeners in mechanical wood, sulfite, soda, and sulfate pulp mills, the water containing finely divided unbleached fibers. The loss of fiber at this point is from 4 to 5% of the total production in mechanical wood pulp mills, and from 2 to 3% in chemical wood pulp mills.

6.—Wash water from the white stock thickeners of rag, soda, and sulfite pulp mills, which contains finely divided bleached fiber. The loss of fiber at this point is from 2 to 3 per cent.

7.—Occasional small quantities of entrained spent liquor from the evaporator in soda and sulfate pulp mills. From a stream-pollution standpoint this waste is negligible.

8.—Black carbon residue left after leaching sodium carbonate from the black ash in soda pulp mills. This amounts to about 15% of the weight of the total pulp production.

9.—Lime sludge (calcium carbonate) containing a small percentage of calcium and sodium hydrates from the alkali room of soda and sulfate pulp mills. The weight of this sludge amounts to about 50% of the total pulp production of the mill.

10.—Lime sludge containing a small percentage of calcium hypochlorite, discharged from the bleach-dissolving department of rag, sulfite, and soda pulp mills. This discharge is proportionately small and can be considered negligible.

11.—Wash water, containing finely divided fibers from the beaters, where the stock is washed and prepared for the paper machines. The loss of fiber at this point is about 1% of the total production.

12.—Excess white water from the paper machines which contains finely divided fibers, clay, and colored water from the pigments and dyes used to color the paper, about 45% of the clay used and 5% of the fiber produced passes away with this white water.

13.—Wash water from the reclaimed paper stock, containing finely divided fiber, clay, and carbon from the removed ink. The waste materials in this water amounts to about 25% of the weight of the old papers treated.

Effects Upon the Receiving Body of Water

Finely divided particles of bark and sawdust from the wood-preparing room and chipper, as well as black ash residue from the soda pulp mills and entrained fibers from the various steps in the manufacture of pulp and paper, will not have any deleterious effect on the sanitary conditions of the stream. However, they will have a tendency to increase the turbidity, and render the stream less pleasing to the sight. In addition, there is a possibility of these materials affecting aquatic life by entering the gills of fish and, through settling on the bed of the stream, affecting the spawning. On the other hand, fish have been known to thrive at the mouth of small streams copiously charged with such effluent.

Spent liquor from sulfite mills, on account of its dark color, will have a tendency to impart a brown coloration to the stream. In addition, the liquor is charged with sulfur dioxide which, being a de-oxidizing agent, will reduce the dissolved oxygen content of the stream.

Spent liquor from soda and sulfate pulp mills will slightly increase the alkalinity content of the stream, but has no deleterious effect, either from the standpoint of sanitation or aquatic life. It will, however, impart a brown coloration and frothy appearance to the stream, making it appear more harmful than it really is.

Lime sludge when discharged into the stream will increase the turbidity. Being composed of lime salts, the hardness of the water will be increased, decreasing its fitness for boiler purposes. It will not affect aquatic life, and will improve the sanitary condition of the water. This discharge has been known to improve greatly the waters in some localities where they are subjected to acid discharges from manufacturing plants.

Clay discharges will increase the turbidity of the stream, but otherwise are entirely inactive. After a rain storm many times the quantity of clay

that could possibly be discharged from paper mills is naturally washed into the streams by the surface water. This waste material, therefore, should not be considered from a stream-polluting standpoint.

The lime sludge from the bleach-dissolving department which contains a small percentage of calcium hypochlorite will increase the turbidity, but has no deleterious effect. However, the presence of free chlorine will improve the sanitary condition of the water.

Assimilating Power of Streams

Although, to the writer's knowledge, no concrete study has been made relative to the amount of pulp and paper mill effluent a stream of a given flow can inoffensively assimilate, it is known from observations that the streams are very effective in this capacity.

When reviewing the points at which waste materials might enter the stream from pulp and paper mills, the worst possible conditions were cited, that is, a mill operating entirely for production without any reference to economy of operation or removal of waste materials from the effluent, as was true before the days of industrial research chemistry.

The pulp and paper companies of the United States, through their research departments, have spent millions in developing uses for, and devising means of, disposal of waste materials from the mills. In addition, the Technical Association of the Pulp and Paper Industry has done considerable work toward eliminating stream pollution, and, at the same time, curtailing waste in the industry.

There are three phases to be considered in discussing waste in pulp and paper mill effluent: First, the improvement of the condition of the stream; second, the re-use of the materials removed; and, third, clarifying the waste waters to such a degree that they can be re-used in the mill and thus conserve the use of fresh water. This last phase is very important for mills dependent on small streams for water supply, which streams are very low during the summer.

Owing to the extensive investigations made by the pulp and paper manufacturers, many of the problems relative to waste in the industry have been solved, and the others are rapidly approaching a satisfactory solution.

Known Methods for Elimination of Waste and Improvement of Streams

Bark from the wood-preparing room is pressed to remove excess moisture and burned in Dutch ovens for its fuel value.

Sawdust from the chipper room is burned either in Dutch ovens or rotary incinerators for its fuel value.

Dust from the rag department has found a market with manufacturers of roofing paper and other similar products.

Improvements have been made in the methods of washing brown stock in soda pulp mills, so that at present the quantity of colored water entering the stream from this source has been reduced to the minimum. If the cycle washing system or continuous filters are used, no washings enter the stream, the drain line being locked.

Waste water from brown and white stock thickeners in rag, mechanical wood, sulfite, soda, and sulfate pulp mills is being passed to efficient save-alls where as much as 98% of the suspended fiber is being removed before passing the water to the stream, or re-using it in the system.

At present, lime sludge from soda and sulfate pulp mills is passed to filters, where the excess water is removed, and thence to rotary dryers, or incinerators, where the lime is either dried, prepared and marketed as agricultural lime and whiting, or re-burned and used again to causticize more soda.

Wash water from the beaters and excess white water from the paper machines is at present being passed to efficient save-alls where as much as 98% of the suspended fibers and 92% of the suspended clay is removed before passing the water to the stream or re-using it in the system.

Unsolved Problems—Waste Sulfite Liquor

Although more money has been spent and more research work conducted to find methods for the disposal of waste sulfite liquor than on any other pulp or paper mill waste, it is still considered one of the unsolved problems. Considerable progress has been made toward reaching a practical solution. Some companies have been able to dispose of some of the waste sulfite liquor for core binder, road binder, tan extract, linoleum cement, and a few other uses, which only consume a very limited quantity of the supply to be had, should all the sulfite pulp companies re-claim this liquor.

It has been contended that the liquor can be evaporated and burned in boiler plants for its fuel value, and this has been tried in several mills; but as the heat value of the organic solids is about 8 000 B.t.u. per lb., and it costs about $5.50 per ton to evaporate it to the point at which it can be burned, this outlet does not appear to be very promising. Some coal companies have found it profitable to briquette hard coal dust by means of waste sulfite liquor and are using the liquor from two mills. When all coal companies reach this same conclusion, they will be in a position to use all the waste liquor at present entering the streams from sulfite pulp mills. This appears to be the not far distant solution to this problem of waste sulfite liquor disposal.

Spent Liquor from Soda and Sulfate Pulp Mill Evaporators

There are times when a small quantity of spent liquor will pass the vacuum pump of the evaporators in soda and sulfate pulp mills. Evaporator manufacturers have improved their product to such an extent that the liquor entering the stream from this point has been reduced to the minimum; when it does occur it only imparts a frothy appearance to the surface of the stream without any deleterious effect.

Carbon Residue from Soda Pulp Mills

At present, some of the black ash residue from soda pulp mills is being prepared and marketed under the names of Filchar, Nuchar, "Superchar", and other trade names, some of it being activated and used as a decolorizing agent. A limited quantity is used as an adulterant for lamp black, and some has been used for the case-hardening of steel.

The outlet for these various purposes is limited, and the disposal of the greater bulk of this waste is still an unsolved problem. However, a furnace has recently been designed and experiments are being conducted for completely burning the carbon in reclaiming the sodium carbonate, thus utilizing its fuel value in the steam boiler plant. If successful, this will be the future method for the recovery of sodium carbonate in soda pulp mills, and will eliminate waste carbon from the mill effluent.

Lime Sludge from Bleach-Dissolving Plants

There has been no satisfactory means for the disposal of lime sludge from bleach-dissolving plants; but as most mills have resorted to the use of liquid chlorine, this discharge is very small, and as it contains a small quantity of calcium hypochlorite, it will act as a germicide.

Pigments and Dyes in Paper Machine Waste Water

There is no known practical method by which the color can be removed from waste water from paper machines when dyes or pigments have been used to produce a high colored sheet. The quantity of such water, however, is small and has no deleterious effect on the stream.

Wash Water from Reclaimed Paper Stock

Numerous experiments have been conducted and considerable money has been expended in endeavoring to dispose of the waste materials washed from de-inked paper stock. It is very difficult to handle this waste material with any form of filter, and as it has so little value, due to the ink carbon content, when reclaimed, the problem of economically handling it is as yet unsolved; but paper manufacturers are optimistic regarding a satisfactory solution in the near future.

Pulp and paper manufacturers are doing everything consistent with operating conditions to improve the waters of the streams. Committees have been formed in some of the States, consisting of State engineers and technical representatives of the various pulp and paper companies, so that they can cooperate in reaching an early solution of the problems. In addition, the President of the American Paper and Pulp Makers' Association has recently appointed a National committee to deal with these problems.

STUDIES ON TANNERY WASTE DISPOSAL

By Wilhelm Howalt,* M. Am. Soc. C. E., and Edwin S. Cavett,† Esq.

Synopsis

This paper presents the results obtained in a two-year investigation of the disposal of tannery wastes. The investigation is being conducted by the Tannery Waste Disposal Committee of Pennsylvania, which is presided over by the Chief Engineer of the Department of Health of Pennsylvania, and is composed of representative engineers and chemists from the tanning interests of that State.

The wastes used throughout the investigation were produced by a medium-sized sole leather tannery at which the vegetable tanning process was used. The study is presented in this paper in four main parts, as follows:

Part I.—Laboratory study of the individual wastes, including chemical analysis of each.

Part II.—Laboratory study of the wastes to find methods of treating by mixing in various combinations, by chemical treatment, etc., to produce reduction of color, of suspended solids, of oxygen demand, etc. From the results obtained a number of treatments were developed and selected for full-scale experimentation.

Part III.—Experimental plant: The experimental plant located at Instanter, Pa., is equipped to operate at full scale. The selected treatments were experimented with in this plant, studies being made by analysis of the various effluents and sludges. All operating data, such as volumes of wastes, effluents and sludges produced, chemical consumption, etc., were collected.

Part IV.—The manner in which the various full-scale treatments affected the receiving body of water was determined by analyzing the river water at certain places for a distance of 12 miles below the tannery. The results are presented in tabular form.

Conclusions are drawn concerning the facts presented and their relation to the general problem of tannery waste disposal.

Introduction

The pollution of streams by industrial wastes has been recognized for many years as an impending problem to the use of water supplies for domestic and municipal purposes. Many industries produce putrescible and deleterious wastes. Some of the more important producers of such wastes are: Coal mines, oil refineries, gas-works, water-gas and coal-gas, chemical works, coke ovens, creameries, tanneries, distilleries and breweries, canneries, municipal sewage, pulp and paper mills, and tin-plate, tubing, and galvanizing works. Many of these industries lately have appreciated the fact that the treatment of industrial wastes, regardless of whether marketable or useful products can

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be recovered from them, is a just charge against their processes. This more reasonable attitude has resulted both from economic conditions and from increasing power given the State and municipal authorities regarding stream pollution.

The importance of stream pollution is emphasized by the fact that almost every State has such a problem and that legislative and constructive measures for its prevention have been steadily increasing.* Practically every State and many municipalities have some law dealing with the pollution of the waters within their jurisdictions.†

In Pennsylvania this subject comes within the jurisdiction of the Health Department and the Sanitary Water Board. In general, this State forbids the discharge into streams of wastes deleterious or detrimental to public health.† Discharge of sewage is forbidden except under permit. Discharge of wastes detrimental to fish is prohibited unless all practicable means are used to prevent pollution. The tanneries are included in these rulings, compliance with which means either the withholding of the wastes or effecting some treatment thereof. Obviously, treatment is the only way open to the tanneries because it would be practically impossible for them to withhold the wastes entirely.

The importance is further emphasized by the magnitude of the tanning industry. The value of all leather tanned in the United States is, roughly, $500 000 000 per year. Probably 50 000 men are employed in the tanneries as wage earners. The hides and skins from the following animals are tanned annually into leather in this country:

Cattle	25 000 000
Calves	15 000 000
Goats	40 000 000
Sheep	33 000 000
Other animals	7 000 000

The Census of Manufacturers of 1921 ranks leather and its finished products—boots, shoes, etc.—as fifth in importance, and the relative position of States producing leather, according to the value of the product, is as follows:

(1) Pennsylvania.
(2) Massachusetts.
(3) New York.
(4) Wisconsin.
(5) New Jersey.
(6) Illinois.
(7) Michigan.
(8) Delaware.
(9) Ohio.
(10) Virginia.
(11) North Carolina.
(12) West Virginia.
(13) California.
(14) Tennessee.

Just what position, on the list of industrial wastes, tannery waste would occupy is a matter of conjecture, although the general public usually considers it the source of obnoxious odors and unsightly, colored, and poisonous liquids, and, being governed thereby, would probably rate it among the worst.

* U. S. Public Health Report, January 15, 1926.
† 69th Cong., H. R., Doc. No. 417.

The problem of tannery waste disposal has never been adequately solved to the satisfaction of interested parties, although the literature contains many references to tannery waste disposal.* Most of them, however, are descriptions of specific treatments and disposal plants and do not contain many data.† No account has been found of an investigation which would presume to set forth any general principles governing the treatment of these wastes.

Realizing the importance of studying the problem from the viewpoint of the industry as a whole, rather than from that of the individual tannery, a committee was formed of engineers and chemists representing the tanning interests of Pennsylvania and having for its Chairman the Chief Engineer of the Department of Health of that State. This Committee, known as the Tannery Waste Disposal Committee of Pennsylvania, has been functioning uninterruptedly since 1924. The work is being financed by the tanning interests of the State and by the State of Pennsylvania, the money to be expended for laboratory studies, construction, and operation of a full-scale treatment plant, and for river studies. It is the final intention of this Committee to publish a report covering various progressive steps of treatment in order that there may be available an answer for every tannery working under its particular conditions both as to size, process, surroundings, and size and nature of the receiving body of water. It is the object of this paper to describe the results to date.

The investigation was conducted in the following sequence:

1.—A study was made of the characteristics of the individual wastes.
2.—Experiments were performed to find methods of treatment for the wastes.
3.—Full-scale operations were carried out on selected methods of treatment.
4.—Effect of these treatments was determined on the receiving body of water.

The individual wastes were studied by means of chemical analysis. Information of this kind served several purposes. It provided a means of classifying the wastes as alkaline, or acid, as to color, and as to process, such as beam house, tan yard, and scrub house. It also provided a means of establishing an order of degree of putrescibility. In practically all cases, these classifications had a direct bearing on the planning and the undertaking of the laboratory investigations for methods of treatment. For these laboratory investigations the wastes were mixed in a carefully planned sequence of combinations and studied as to rates of settling, color reduction, solids reduction, etc. This particular end of the investigation has not been thoroughly and exhaustively completed as yet, but for the type of full-scale operation available and for the degree of treatment desired at this time, a sufficient number of methods have been developed.

Several selected treatments, which, from the laboratory viewpoint, seemed promising, were chosen by the Committee as worthy of trial on full scale. These treatments were carefully studied in all respects from the practical and

* Bibliography of Tannery Waste Treatment, by P. A. Esten, *Journal*, Am. Leather Chemists Assoc., October, 1911.

† *Public Health Bulletin No. 100*, November, 1919.

economical standpoint as well as from the degree of success and efficiency obtained. Finally, the effect of the effluents on the stream, after treatment and after they had been discharged to the stream, were studied by analyses of the receiving body of water, sampled at five carefully selected stations along the 12 miles of stream immediately below the point of discharge.

The laboratory investigation, the routine analyses of the river, and treatment plant samples were made in the main laboratory of the Elk Tanning Company, Ridgway, Pa. The treatment plant was at the Instanter Tannery of the same Company, at Instanter, Pa., and the river water samples were taken over a course of 12 miles on the East Branch of the Clarion River, between the Towns of Instanter and Johnsonburg, Pa. There were several excellent reasons for selecting this tannery and this location.

First.—The Instanter Tannery is a typical example of a sole leather tannery working an average number of hides and producing an average volume of wastes. These facts were confirmed by a survey of all the tanneries practicable. A questionnaire was sent to all the tanneries in Pennsylvania. The authorities of Ohio and West Virginia, having signified their intention of co-operating with the Tannery Committee in Pennsylvania, were also supplied with questionnaires for distribution in their respective States. These questionnaires asked for specific information regarding the pounds of hides worked and the volumes of waste produced per day from the various operations of the tanning process. Fifty-one tanneries, in ten States, returned answers. Tanneries in other States sent replies by reason of the fact that several members of the Committee represented companies having tanneries in these States which were likewise supplied with the questionnaires. The 51 tanneries were distributed as follows: 24 in Pennsylvania; 10 in West Virginia; 8 in Ohio; 2 in New York; 2 in Wisconsin; 1 in Maryland; 1 in Michigan; 1 in Kentucky; 1 in Tennessee; and 1 in Virginia. Of the 51 reported, 45 manufacture sole and belting leather. There was great variation in the amount of waste produced, which, based on 100 lb. of hide, ranged from 230 to 1 790 gal. per day, due to scarcity or abundance of water supply. The Instanter Tannery, with 617 gal. per day per 100 lb. of hide, is a good average.

Second.—The tannery is situated just at the confluence of two small streams, neither of which receives any pollution from sewage or industrial waste. Likewise, below this point of confluence, and for 12 miles down stream, there is no incoming sewage or industrial waste, the tannery being the only user of the river within this distance.

Third.—It was allowable for the tannery to pollute the river at will within the 12-mile distance in order to experiment with the various degrees of treatment.

Fourth.—The convenience and almost necessity of having the laboratory so near the whole field of action made the location at Instanter unusually favorable to the investigation.

Thus, an ideal situation obtains for making a direct study of the receiving body of water to determine the effect of dilution, temperature, aeration, and the various other factors influencing the wastes in their course down the river.

Origin and Description of Individual Wastes

Instanter Tannery is a sole leather tannery, and uses the vegetable process, which is the most common of all in use at present for tanning heavy leather. The finishing processes for such leathers are somewhat different in various tanneries, but, as a rule, very little wastes are produced and are negligible as compared with the whole volume of the other wastes. At Instanter the bulk of the wastes studied were produced from three departments of the tannery, namely, the beam house, the tan yard, and the scrub house. The individual wastes produced from these three places and their volumes per day are as follows:

Wastes from:	Gallons per day.	
Beam House:		
Soaks	9 600	
Limes	2 900	
Hot water	2 900	
Unhairing machine	3 100	
Wash wheel	25 800	
Fleshing machine	7 200	
Float box	4 700	
Green stock pools	43 600	
Brown hair wash	14 300	
White hair wash	1 200	
Total		115 300
Tan Yard:		
Spent tan liquor	7 800	
Total		7 800
Scrub House:		
Acid bleach	1 700	
Soda bleach	1 700	
Acid water	1 700	
Total		5 100
Floor Wash:		
Floor wash from all departments		1 800
Grand total		130 000

The wastes as produced from the three departments can be classified as continuous, or those discharging continuously during the day, and as intermittent, or those released at a certain time once each day.

The three principal departments of the average tannery of this kind are the beam house, the tan yard, and the finishing department, of which the scrub house is a part. The beam-house operations prepare the hide for tanning; the hides are tanned in the tan yard; and the leather is finished in the finishing department. The description of the tanning process at Instanter, therefore, naturally starts with the beam-house operations.

Beam House

Soaking.—Heavy, green, salted, cow and steer hides are first placed in vats containing water which is discharged twice a week. They hang in these vats for one day, after which they are moved forward into another vat. The water is discharged once a week. The hides are moved in this manner every day during the 3 to 5 days comprising the soaking period. As the hides move forward from vat to vat, they become cleaner and softer, so that generally the last soak waters can be used longer than the first. This accounts for the fact that the first soak is renewed twice a week and the others only once a week. A stream of clean water runs into the vats day and night. Thus, the waste soak waters will contain the dirt, dung, blood, and considerable quantities of hair and sodium chloride as major impurities, and are usually a dark, dirty, olive green color. They will vary in strength from day to day, making it possible to discharge four different strengths. During the normal operations, three vats are discharged daily, amounting approximately to 9 600 gal. per day.

Liming.—The hides, having been thoroughly cleaned and softened in the soak vats, are reeled into vats containing a super-saturated solution of calcium hydroxide, to which a small percentage of sodium sulfide has been added. In the lime solution the hair is loosened by the dissolution of the outside layer of the hide—the epidermis—and the cementing substances in the hair follicles. The process requires usually from 4 to 6 days, depending on the kind of hides. The lime solutions are used from 1 to 3 weeks, being strengthened daily with fresh lime. When the lime solution has been in use so long that it will no longer give the proper action, it is discharged as a waste liquor. At Instanter, the supernatant lime liquor is discharged to the sewer, and the lime sludge, which accumulates in the bottom of the vats, is shoveled out and hauled away. It is not washed into the discharge sewers. The principal impurities of the lime liquors are dirt, hair, and dissolved organic matter of a putrescible nature. It has a typical milk-of-lime appearance with a slight bluish color and contains much lime in suspension. There are six different kinds, or strengths, of limes resulting from the fact that the hides dirty the first limes more than the later ones and for other reasons having no bearing on the problem at hand. This waste amounts to about 2 900 gal. per day.

Hot Water.—The last lime solution, called the hot-water solution, differs from those preceding it in that it is not strengthened after it is once made up and that it is maintained at a temperature of about 90° Fahr. This solution is never used longer than three days, when it is discharged and made up new. It has the general appearance of the lime wastes and is discharged in the same manner. It amounts to about 2 900 gal. per day.

Unhairing Machine, Brown Hair Wash, White Hair Wash.—When the hides are removed from the hot water, the hair has been loosened and can easily be removed. This operation is performed on the unhairing machine by a heavy, revolving, roller knife sweeping down over the hide and brushing the hair off. A small quantity of water trickles over the knife during the time it is in operation and washes the hair into a trough below, down which it floats into a vat where it is automatically removed. This hair is the brown hair,

which is then collected and washed in a stream of clean, running water, the overflow from which unites with that from the aforementioned vat to form the combined unhairing machine and brown hair wash wastes. The white hair, having been removed by hand previous to the unhairing operation, is washed separately, and furnishes the white hair waste. The unhairing machine, brown hair wash, and white hair wash produce daily wastes amounting to 3 100, 14 300, and 1 200 gal., respectively. The appearance of all three wastes is much the same, having a slight turbidity due to lime and containing some fine hair which has escaped the screens.

Wash Wheel.—The hides having been unhaired and fleshed still contain much lime. To remove this lime the hides are placed in large wheel drums and drummed while a stream of clean water is flushed on them. The pounding and tumbling in the wheel works the lime out and the water carries it away. The wash-wheel waste is quite large in volume, amounting to about 25 800 gal., and contains mainly lime and small pieces of flesh. It is slightly turbid and has the appearance of milky water.

Fleshing Machine.—After the hides have been unhaired and drummed, the flesh, more or less of which is always present on the back of the hide, is removed by a machine similar to that used for unhairing. The fleshings drop down behind the machine and are floated in water into a vat from which the water passes to the sewer. This wash water carries with it small pieces of flesh and a considerable amount of fine particles of fleshings and fat in suspension. This is highly putrescible waste, has a cream-colored appearance, and averages about 7 200 gal. per day.

Float Box.—The float box is nothing more than a long vat with a stream of clear water running in at one end and overflowing at the other. Its purpose is to keep the unhaired, fleshed, and washed hides in a water-soaked condition from the time they leave the fleshing machine until they are ready to be trimmed and hung in the clear-water pools. The float-box water contains very little lime and a small amount of suspended matter composed principally of small pieces of hide substance. It has the appearance of clear water with light-colored particles suspended in it, and amounts to about 4 700 gal. per day.

Green Stock Pools.—Into green stock, or clear-water, pools the now thoroughly washed and cleaned hides are placed to await the tanning process. Clean water is run into these vats continuously, and the waste water contains very few impurities and shows practically no discoloration or suspended matter. It has a slight turbidity and amounts to about 43 600 gal. per day, or one-third the total daily waste from the tannery.

This process concludes the beam-house operations, which furnish the bulk of the daily wastes from the tannery.

Tan Yard

Rockers.—The hides are taken from the green stock pools and placed in the oldest and weakest tanning liquor in the system. The tanning liquors are composed chiefly of infusions of chestnut wood, quebracho wood, and hemlock bark. This liquor, containing only about 0.5% of tannin, is known as the "tail rocker". The name, "rocker", is attributed to the fact that the

TABLE 5.—CHEMICAL ANALYSES OF INDIVIDUAL WASTES.

	Wastes.	Gallons per day.	Specific gravity, 20° cent.	Total Solids.		Suspended Solids.		Fixed Residue.		Volatile Solids.		Total Alkalinity, CaO.	
				Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.
1*	Soaks, first	9 600	1.0173	23 820	1 940	1 888	154	21 233	1 729	2 587	211	689	56
	Soaks, second		1.0022	3 101	249	383	31	910	73	2 191	176	140	11
	Soaks, third		1.0022	3 030	243	262	21	1 556	125	1 474	118	392	31
	Soaks, fourth		1.0028	4 400	353	1 002	80	1 803	145	2 597	209	168	13
			1.0040	7 522	605	630	51	3 044	245	4 478	360	279	22
			1.0097	14 250	1 152	701	57	13 240	1 070	1 010	82	416	34
2*	Limes, first (6 days)	2 900	1.0100	18 456	451	8 193	200	14 662	358	3 794	93	13 186	322
	Limes, second (2 weeks)		1.0100	18 780	458	10 254	251	14 491	354	4 289	101	7 030	172
	Limes, third (2 weeks)		1.0090	11 949	292	1 954	48	8 504	208	3 445	84	2 514	61
	Limes, fourth (3 weeks)		1.0156	25 270	621	13 305	327	24 896	612	374	9	21 434	600
	Limes, fifth (3 weeks)		1.0145	14 987	368	4 800	118	†	†	†	†	14 844	364
	Limes, sixth (2 to 3 weeks)		1.0150	23 432	575	7 605	187	18 463	453	4 969	122	6 331	155
			1.0128	28 764	705	17 003	417	27 834	682	920	23	28 766	705
			1.0140	23 026	565	6 134	150	16 446	403	6 580	161	5 730	141
			1.0122	19 332	473	8 352	204	13 251	324	6 081	149	16 403	402
3*	Hot water	2 900	1.0045	8 235	200	2 477	60	6 292	153	1 943	47	6 364	155
			1.0096	8 653	211	3 312	81	4 857	119	3 796	93	3 444	84
			1.0075	9 965	243	3 481	85	†	†	†	†	3 836	93
4	Unhairing machine	3 100	1.0000	1 906	49	1 102	28	908	23	998	26	751	19
5	Wash wheel	25 800	1.0000	1 620	349	448	96	524	113	1 096	236	224	48
6	Fleshing machine	7 200	1.0000	3 557	214	2 571	154	†	†	†	†	112	7
7	Float box	4 700	1.0000	276	11	146	6	†	†	†	†	28	1
8	Green stock pools	43 600	1.0000	385	140	94	34	135	49	250	91	112	41
9	Brown hair wash	14 300	1.0035	2 434	291	984	118	876	105	1 558	187	112	13
10	White hair wash	1 200	1.0016	2 212	22	909	9	1 368	13	904	9	420	4
11*	Spent tan liquor	7 800	1.0152	34 255	2 262	817	54	5 079	335	29 176	1 927	.....	.....
12*	Soda bleach	1 700	1.0171	30 105	434	2 565	37	7 103	102	23 002	332	4 021	58
			1.0190	33 720	477	2 983	43	8 169	118	25 551	359	3 720	37
13*	Acid bleach	1 700	1.0233	36 349	527	20	...	5 992	87	30 357	440	.....	.....
			1.0256	41 158	599	643	9	6 728	98	34 434	501	.....	.....
14*	Acid water	1 700	1.0110	17 242	247	19	...	2 813	40	14 429	207	.....	.....
			1.0140	21 361	307	147	2	3 629	52	17 732	255	.....	.....
		128 200	.....	.....	6 342	.....	921	.....	2 261	.....	3 868	.....	669

* Nos. 1, 2, 3, 11, 12, 13 and 14 are intermittent wastes. † No analysis made. ‡ Composite sample.

TABLE 5.—(*Continued*).

TABLE 5.—(*Continued*).

	Wastes.	Gallons per day.	Specific gravity, 20° cent.	Sulfuric Acid.		Sodium Chloride		Total Nitrogen.		Hide Substance.		Five-Day Oxygen Demand, 20° Cent.	
				Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.
1*	Soaks, first	9 600	1.0173	.....	.....	19 901	1 621	303	24.7	1 701	139		
	Soaks, second		1.0022	.....	.....	2 263	182	49	3.9	275	22		
			1.0022	.....	.....	3 118	250	43	3.5	243	20		
						1 962	157						
	Soaks third		1.0028	.....	.....	2 792	224	86	8.9	483	39	190‡	15‡
			1.0040	.....	.....	6 215	499	382	30.7	2 145	172	558‡	45‡
						10 677	858						
	Soaks, fourth		1.0097	.....	.....	12 111	979	159	12.9	893	72		
						3 212	260						
2*	Limes, first (6 days)	2 900	1.0100	.....	.....	.....	.....	271	6.6	1 524	37		
	Limes, second (2 weeks)		1.0100	.....	.....	.....	.....	286	7.0	1 608	39		
			1.0090	.....	.....	.....	.....	135	3.3	758	19		
	Limes, third (2 weeks)		1.0156	.....	.....	.....	.....	410	10.1	2 300	57		
	Limes, fourth (3 weeks)		1.0145	.....	.....	.....	.....	516	12.7	2 900	71	3 300‡	80‡
	Limes, fifth (3 weeks)		1.0150	.....	.....	.....	.....	754	18.5	4 234	104	1 510‡	37‡
	Limes, sixth (2 to 3 weeks)		1.0128	.....	.....	.....	.....	604	14.8	3 395	83		
			1.0140	.....	.....	.....	.....	1 135	27.8	6 380	156		
			1.0122	.....	.....	.....	.....	696	17.0	3 908	96		
3*	Hot water	2 900	1.0045	.....	.....	.....	.....	361	8.8	2 030	49	526‡	13‡
			1.0096	.....	.....	.....	.....	122	3.0	686	17	2 255‡	55‡
			1.0075	.....	.....	.....	.....	538	13.1	3 029	74		
4	Unhairing machine	3 100	1.0000	.....	.....	.....	.....	80	2.1	448	12	140	3.6
												136	3.5
5	Wash wheel	25 800	1.0000	.....	.....	.....	.....	105	22.6	590	127	502	108
												462	95
6	Fleshing machine	7 200	1.0000	.....	.....	.....	.....	105	6.3	590	35	828	50
												310	19
7	Float box	4 700	1.0000	.....	.....	.....	.....	†	†	†	†	44	1.7
												6	0.3
8	Green stock pools	43 600	1.0000	.....	.....	.....	.....	17	6.3	97	35	3	1.2
9	Brown hair wash	14 300	1.0035	.....	.....	.....	.....	79	9.5	444	53	138	16
												328	39
10	White hair wash	1 200	1.0016	.....	.....	.....	.....	71	0.7	399	4	431	4
												82	0.8
11*	Spent tan liquor	7 800	1.0152	1 206	80	.....	.....	89	5.9	500	33	5 725	373
												4 340	283
12*	Soda bleach	1 700	1.0171	.....	.....	.....	.....	34	0.5	192	3	3 144	47
			1.0190	.....	.....	.....	.....	23	0.3	127	2	2 715	38.5
13*	Acid bleach	1 700	1.0233	17 597	255	.....	.....	52	0.8	202	4	1 700	24
			1.0256	18 633	271	.....	.....	58	0.8	326	5	1 655	23
14*	Acid water	1 700	1.0110	5 186	74	.....	.....	34	0.8	191	3	1 500	21
			1.0140	5 866	84	.....	.....	41	0.6	228	3	1 025	15
		128 200	.....	.....	422	.....	743	.....	94.8	.....	531	.....	706.4

* Nos. 1, 2, 3, 11, 12, 13 and 14 are intermittent wastes. † No analysis made. ‡ Composite sample.

frames upon which the hides are suspended in the vats are rocked slowly up and down. After remaining in the tail rocker one day, the tail-rocker liquor is drained off, as waste or spent tan liquor, and is run to the sewer. This waste is highly colored and represents the worst single waste from the tannery as to color, solids, oxygen demand, etc. It amounts to about 7 800 gal. per day, has a reddish, chocolate brown color, and is slightly acid in character. The hides are then passed from the rockers to vats called "layers", containing heavy tanning liquor in which the tanning is completed. No liquor from the layers finds its way to the waste. The only waste produced in the tan yard is that from the tail rocker.

Finishing

Scrub House.—In this department the leather from the layers is bleached and impregnated with oils, concentrated tanning liquor, sugar, salts, etc., depending on the character of leather desired. Except for the bleach, none of the other operations furnishes waste. The bleach liquors are made up daily and discharged to the sewer at the end of the working day. The bleaching proper takes place in a solution of sodium carbonate, after which the leather is given a dip in a weak sulfuric-acid solution to neutralize the excess sodium carbonate, and, finally, a dip in water. The three wastes are designated soda bleach, acid bleach, and acid-water bleach, each furnishing 1 700 gal. per day.

Part I.—Chemical Analyses of Individual Wastes

The object of carefully analyzing the individual wastes was to bring out the differences existing between them so that they might be classified physically and chemically according to their several attributes. It was found that the volumes of the wastes varied somewhat from day to day, due mainly to the use of different quantities of water in the processes, but that the actual constituent parts of the wastes remained fairly constant. In some instances, therefore, more than one analysis was made of a given waste to note this effect. A study of these analyses threw considerable light on the subject of treating the wastes, and on the accurate planning of the preliminary laboratory work. These analyses are shown in Table 5.

The hide substance was calculated from the total nitrogen, assuming that the nitrogenous matter represented that of hide matter and that it equalled 17.8% thereof. Only three of the wastes were acid, namely, the spent tan liquor, the acid bleach, and the acid-water bleach. Combined, these three weak acid wastes amounted to only approximately 11 000 gal., which was 8.5% of the whole volume of waste and would have little influence toward neutralizing the combined volume of the other wastes, due to their high alkalinity. The remainder of the wastes, except the soda bleach, originated in the beam house and were alkaline, due to the lime used there; that is, all the wastes discharging from the beam house following the limes were contaminated, so to speak, with the lime carried over by the hides from the lime vats. Approximately 50% of the alkalinity of the soaks, however, was due to free ammonia, whereas the alkalinity of the soda bleach was due to sodium carbonate.

Referring to Table 5, the wastes listed as intermittent represent by far the worst. They are all vat wastes, or those which result when the solutions in

the vats, having been used over and over, no longer function properly and are discharged. In general, the characteristics of the intermittent wastes were that they contained the most total solids, suspended solids, and color, had the greatest oxygen demand, and were responsible for most of the variation in volume from day to day.

All the continuous wastes were alkaline; and as they originated from a washing operation were neither as putrid nor as concentrated as the intermittent wastes. This fact is interesting, because it means that at some tanneries on large streams these wastes can be disposed of without treatment. On the other hand, where it is necessary to treat all the wastes, these would serve ably for dilution purposes. In this investigation, however, all the wastes were used in the experimentation, as obviously no such investigation would be complete unless all were included in the study.

TABLE 6.

	First Set.						Second Set.					
	Oxygen demand, in parts per million.	Order.	Percentage quantity units, in pounds of oxygen.	Quantity units, in pounds of oxygen.	Order.	People = $\frac{Q.\ U.}{0.167}$	Oxygen demand, in parts per million.	Order.	Percentage quantity units, in pounds of oxygen.	Quantity units, in pounds of oxygen.	Order.	People = $\frac{Q.\ U.}{0.167}$
*Spent tan	5 725	1	49.3	373.0	1	2 230	4 340	1	43.2	283.0	1	1 695
*Limes	3 300	2	10.6	80.0	3	478	1 510	5	5.7	37.0	7	221
*Soda bleach	3 144	3	6.2	47.0	5	282	2 715	2	5.8	38.5	6	231
*Acid bleach	1 700	4	3.2	24.0	6	144	1 655	4	3.4	23.0	8	138
*Acid water bleach	1 500	5	2.8	21.0	7	126	1 025	6	2.3	15.0	10	90
Fleshing machine	828	6	6.6	50.0	4	299	310	10	2.8	19.0	9	114
*Hot water	526	7	1.7	13.0	10	79	2 255	3	8.3	55.0	3	319
Wash wheel	502	8	14.3	108.0	2	648	462	8	14.5	95.0	2	570
White hair wash	431	9	0.5	4.0	11	24	82	12	0.1	0.8	12	5
*Soaks	190	10	2.0	15.0	9	90	558	7	6.9	45.0	4	269
Unhairing machine	140	11	0.48	3.6	12	22	136	11	0.54	3.5	11	21
Brown hair	138	12	2.10	16.0	8	96	328	9	6.0	39.0	5	234
Float box	44	13	0.22	1.7	13	10	6	13	0.46	0.3	13	2
Total	.....	..	100.00	756.3	..	4 528	.....	..	100.00	653.1	..	3 909

*Intermittent wastes.

Referring to the determination of 5-day bio-chemical oxygen demand given in the last column of Table 5, an interesting arrangement of the wastes is obtained when using this determination for the standard. This arrangement is shown in Table 6.

Two sets of samples of the wastes, shown in Table 6, were collected on two different days. In the first column of each set is given the 5-day oxygen demand at 20° cent., in parts per million. The number in the column, "Order", refers to the order of intensity, No. 1 having the highest oxygen demand, No. 2 the next highest, etc. The figures in the column, "Quantity Units, in Pounds

of Oxygen", are obtained by reducing the parts per million results to a pound basis. These figures represent the pounds of oxygen required in five days by each waste for stabilization. If they are divided by 0.167 in each instance, the pounds are converted into people; that is, the average person produces enough waste material in a day of 24 hours to create a 5-day oxygen demand of 0.167 lb. at 20° cent.* These figures are given under the column, "People". The other column gives the "Percentage Quantity Units".

It is interesting to note that the spent tan liquor, although only representing 6% of the entire waste by volume, accounted for almost 50% of the total oxygen demand; also, that the combined wastes per day represented the sewage from a town of about 4 200 people; that is, for a tannery of 300 hides per day, 1 hide produces waste with an oxygen demand equivalent to that of the sewage of 14 people.

Another point of interest is the wash-wheel waste. It has a relatively low oxygen demand, eighth in order of intensity, but when converted to quantity units assumes a position second only to the spent tan liquor, representing about 14% of the total oxygen demand of the wastes. This is by reason of its relatively large volume of 14 300 gal., or 11% of the total daily wastes from the tannery.

The statement made previously that the intermittent wastes were by far the worst is very strikingly brought out by the figures of Table 6. In both sets of samples the intermittent wastes represent slightly more than 75% of the total oxygen demand, whereas their combined volume is less than 30% of the total daily waste from the tannery.

The principal points of importance learned from the preliminary investigation of the wastes are:

(1) The majority of the wastes were found to be alkaline; only the acid bleach, acid-water bleach, and the spent tan liquor proved to be acid.

(2) In many cases the continuous wastes probably could be disposed of without treatment, or could be used as a diluent in treating the other wastes.

(3) With the exception of the soda bleach, all the alkalinity found was due to lime.

(4) The spent tan liquor and the three bleach liquors contained most of the coloring matter of the wastes.

(5) The intermittent wastes constituted about 75% of the total oxygen demand of the wastes.

(6) The continuous wastes representing more than two-thirds of the total volume of all the wastes accounted for only 25% of the total oxygen demand.

(7) Based on quantity units, the spent tan liquor required about 45% of the total oxygen demand.

Part II.—Laboratory Studies

The object of the laboratory studies was to develop methods of treatment by making a thorough study of all the possible mixtures of the various wastes. In order to make such an investigation it is first necessary to learn the proper-

* "Treatment of Packing-House, Tannery, and Corn Products Wastes," *Industrial and Engineering Chemistry*, October, 1926.

ties of the individual wastes and then to study systematically their behavior when mixed each with the other, then each with two others, etc., until all possible combinations have been tried. The sterilizing, deodorizing, stabilizing, decolorizing, etc., of tannery waste is not a difficult problem as such, but to do all this by merely mixing the wastes in combinations which produce chemical reactions, precipitation, sedimentation, etc., at low cost, throws an entirely different light on the problem. To date more than one hundred such combinations have been investigated.

A limitation defining the proportion of each constituent in the mixtures made it necessary to consider in these laboratory investigations the importance of always mixing the wastes in the same proportions in which they are discharged. Not only a change in the proportion of one or more constituents in a mixture, but also in practice, the time of day of discharge of a given waste, or its variation in amount, strength, and character from day to day, might influence the success of a treatment.

In the preliminary study of the various mixtures, practically the only objectives were rapidity and adequateness of sedimentation, solids removed, and color removed. Whenever an outstanding mixture, with regard to these qualities was obtained, it was studied in greater detail. In other words, unless a mixture proved to be above the average, it was listed as a matter of record as negative information.

The equipment used in the laboratory for the study of sedimentation, color, and general physical characteristics were either 1 000 or 500-cu. cm., graduated glass cylinders. They were of uniform bore and colorless, and a practice was made from the start to use cylinders of the same size throughout a given experiment; that is, the contents in a 500-cu. cm. cylinder were never compared with the contents of a 1 000-cu. cm. cylinder.

Sampling

Throughout these experiments the wastes were always sampled in the same manner, and only fresh samples were used. The soaks, limes, and hot waters were sampled by plunging up the vats just prior to their discharge to the sewer. The unhairing-machine wastes were sampled by taking a quantity from the trough and removing the excess hair. The wash-wheel waste was sampled by placing a bucket beneath the wheel so as to catch a small quantity of the wash water during the whole time the wheel was running, care being taken to prevent the bucket from overflowing. The fleshing-machine waste was sampled in the same manner as the unhairing-machine waste, but large pieces of flesh were removed. The float-box waste was sampled by merely dipping out a pailful. The green stock pools were sampled in the same manner as the float box. The brown hair wash was sampled by placing a bucket near the overflow so as to catch a small portion of the waste over a period of several hours, care being taken not to overflow the bucket. The white hair wash sample was taken from the wash water contained in the white hair washing vat, first plunging up the water and withdrawing a quantity. The soda, acid, and acid-water bleaches were sampled by plunging up the vats just prior to their discharge, and a sample taken from each. The spent tan liquor was sampled by making a

composite sample of the six rocker vats discharged each day. It was not practicable to obtain a sample of the various floor washes which varied so much in quantity and character, and because of their relatively small volume they were neglected.

Treatment of Intermittent Wastes

It has already been stated that the intermittent wastes represent by far the worst group from the tannery. Considerable time, therefore, was spent in experimenting with these seven wastes in whole, in part, and with chemical treatment. The particular result looked for in these treatments was rapidity of settling the settleable solids, and the reduction of color, oxygen demand, and suspended solids.

Experiment No. 14 (Fig. 2) was performed several times in order to note the main reactions between the individual wastes. Unless there was a tendency to settle or a decided change in the two constituents, the mixture was termed "no reaction". Experiments Nos. 2 and 3 (Fig. 3), 4 and 5 (Fig. 4), 7 and 8 (Fig. 5), 11 (Fig. 6), and 12 (Fig. 7), are examples of many such experiments using several combinations of wastes, permitting them to settle, and then combining the resulting supernatants. The object of these experiments was to find ways and means, without chemical treatment, to effect good settlement and to produce light-colored and clear effluents. Experiment No. 1 (Fig. 8), shows the general characteristics of the whole mixture of intermittent wastes.

After a considerable number of experiments had been performed similar to those described, it became evident that to produce a really worth-while effluent and not to complicate a treatment with too many refinements, the number of separate treatments of any of the wastes or combinations thereof should be kept as low as possible. Treatment of a large number of small units materially introduces operating difficulties which are hard to overcome. Also, the cost attending the handling of any treatment consisting of a group of small units usually is prohibitive. From a practical standpoint a mixture of all the wastes in one batch would offer the simplest means for treatment in most cases, and even if additional agents, such as acids, alkalis, coagulants, etc., were required, the ease and simplicity of operation would probably be a deciding factor in its favor.

One of the most difficult problems was to produce a mixture which would settle readily. Several plans were developed which gave encouraging results, but the disadvantage was that several separate units had to be treated individually in order to produce the desired effect. As stated, it became apparent that a paramount requisite was simplicity; that mixing all these wastes together seemed the most promising method for further study. As such, however, as was shown by Experiment No. 1 (Fig. 8), and reproduced many times thereafter, the mixture of the intermittent wastes was alkaline, and there was little or no tendency toward sedimentation. A number of experiments having to do with the treatment of all the intermittent wastes mixed together, therefore, were carried out, using the chemical agents, lime, sulfuric acid, sulfur dioxide, aluminum sulfate, ferrous sulfate, etc. Three typical examples of this study are shown by Experiments Nos. 9 (Fig. 9), 26(*a*) (Fig. 10), and 42 (Fig. 11).

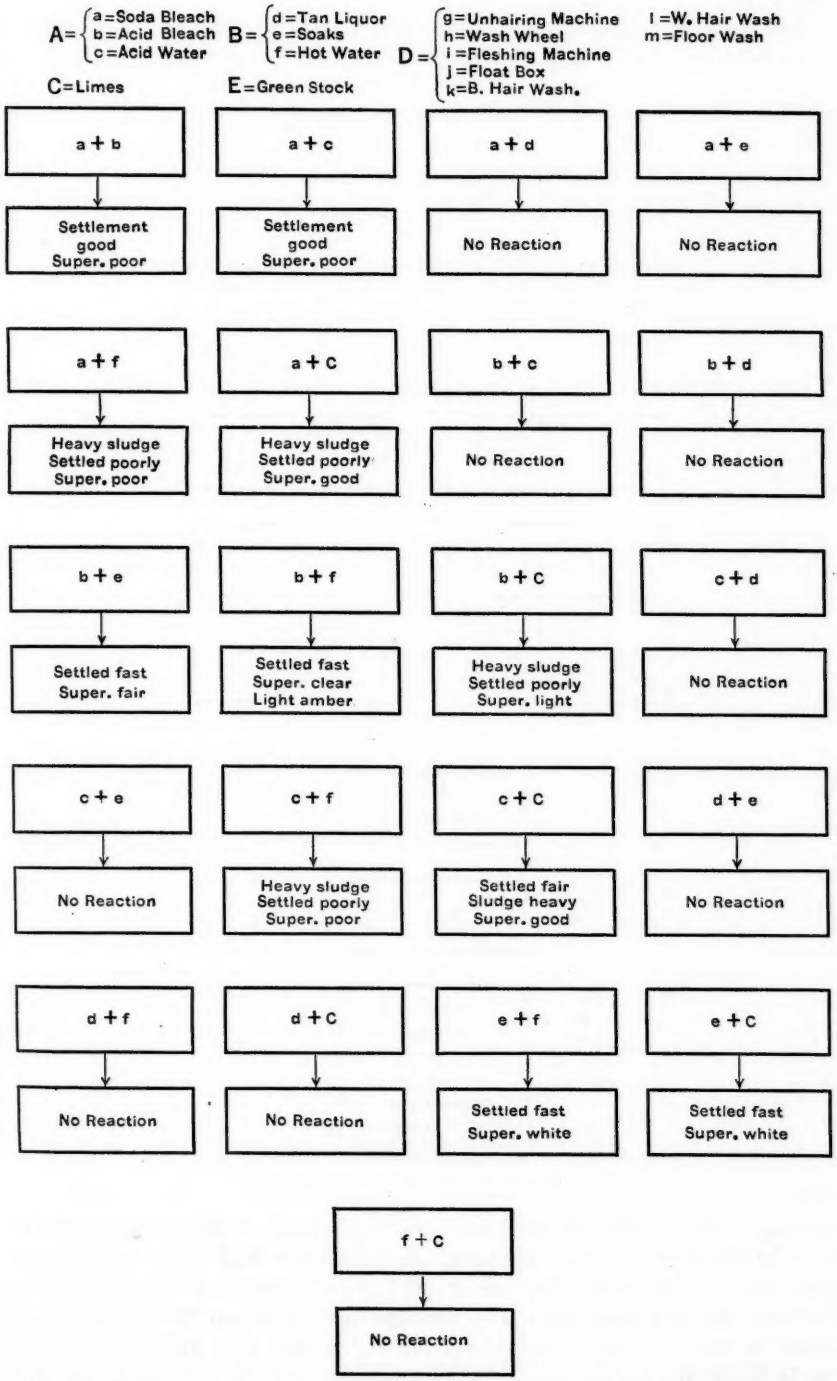

FIG. 2.—EXPERIMENT NO. 2 TO NOTE MAIN REACTIONS BETWEEN INDIVIDUAL WASTES.

The treatment of the intermittent wastes with sulfuric acid was selected as the most promising from all standpoints, because it represents probably the simplest kind of treatment and the most dependable thus far found. By merely regulating the pH of the mixture to between 5.0 and 6.0, the settleable solids readily settled out in about 3 hours to about 20% of the volume, and the supernatant liquor contained practically no suspended matter. This simple regulation, requiring little acid, had in its favor the advantage of

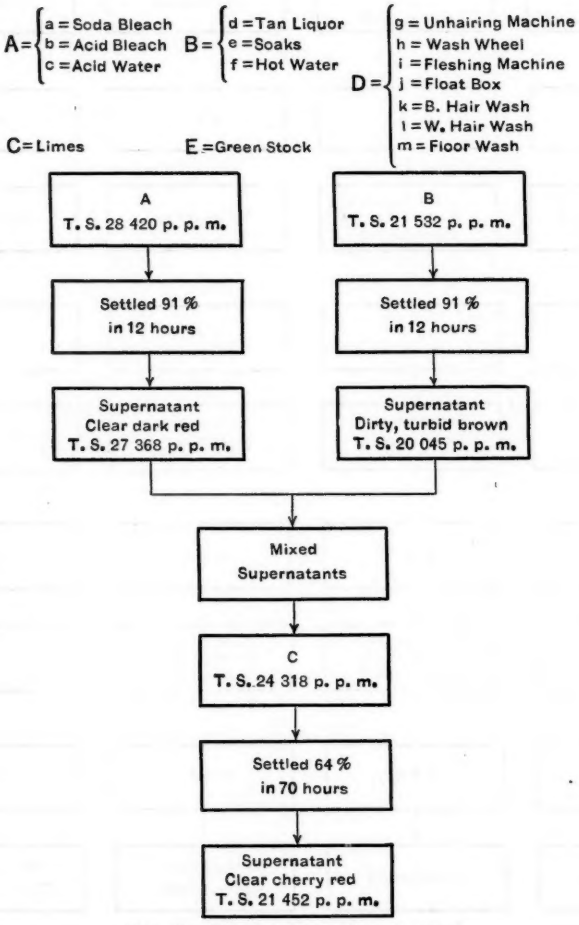
A = a = Soda Bleach
b = Acid Bleach
c = Acid Water
B = d = Tan Liquor
e = Soaks
f = Hot Water
D = g = Unhairing Machine
h = Wash Wheel
i = Fleshing Machine
j = Float Box
k = B. Hair Wash
l = W. Hair Wash
m = Floor Wash
C = Limes
E = Green Stock
A
T. S. 28 420 p. p. m.
B
T. S. 21 532 p. p. m.
Settled 91 % in 12 hours
Settled 91 % in 12 hours
Supernatant Clear dark red T. S. 27 368 p. p. m.
Supernatant Dirty, turbid brown T. S. 20 045 p. p. m.
Mixed Supernatants
C
T. S. 24 318 p. p. m.
Settled 64 % in 70 hours
Supernatant Clear cherry red T. S. 21 452 p. p. m.

FIG. 3.—EXPERIMENTS NOS. 2 AND 3.

producing with unwavering regularity the same kind of effluent from day to day. Variations in amount, strength, and character had little influence on the mixture, for regardless, the suspended matter never failed to settle readily when this pH was attained. The average data obtained from twenty-two separate mixtures of the intermittent wastes, treated with sulfuric acid, are given in Table 7.

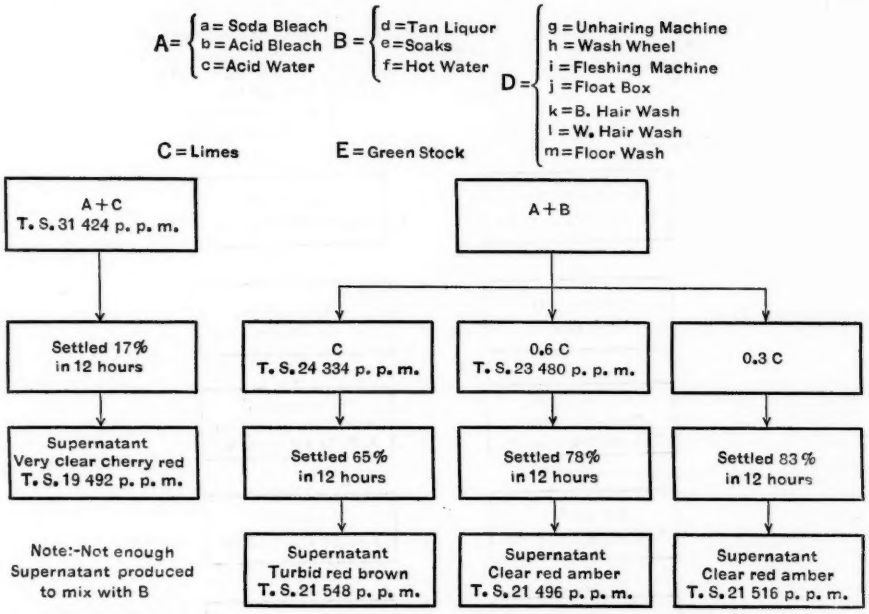

FIG. 4.—EXPERIMENTS NOS. 4 AND 5.

FIG. 5.—EXPERIMENTS NOS. 7 AND 8.

There are probably many cases where this kind of treatment of the most foul wastes would suffice. In such, it would be assumed that the continuous wastes were of such a nature, or the receiving body of such magnitude, that they could be disposed of without treatment.

TABLE 7.—ANALYSIS OF INTERMITTENT WASTES BEFORE AND AFTER TREATMENT WITH SULFURIC ACID.

	Total solids, in parts per million.	Hide substance,* in parts per million.	Organic matter, in parts per million.	5-day oxygen demand, in parts per million.	Color.
Whole mixture	27 090	1 235	15 116	3 302	8 500
Supernatant	24 213	362	12 511	1 274	3 000
Percentage reduced	10.62	70.70	17.23	61.40	64.75

* Average of three samples.

Treatment of Intermittent and Continuous Wastes

In this section of the paper the writers consider not only the intermittent wastes, but, in addition, the continuous wastes, except the green stock. Many experiments were carried out with these wastes in much the same manner as that followed in the preceding section. They were studied first, by mixing them in small units, permitting sedimentation, and eventually combining the supernatants until the final effluents represented all the wastes; then, less complicated methods were tried; and, finally, the number of separate mixtures

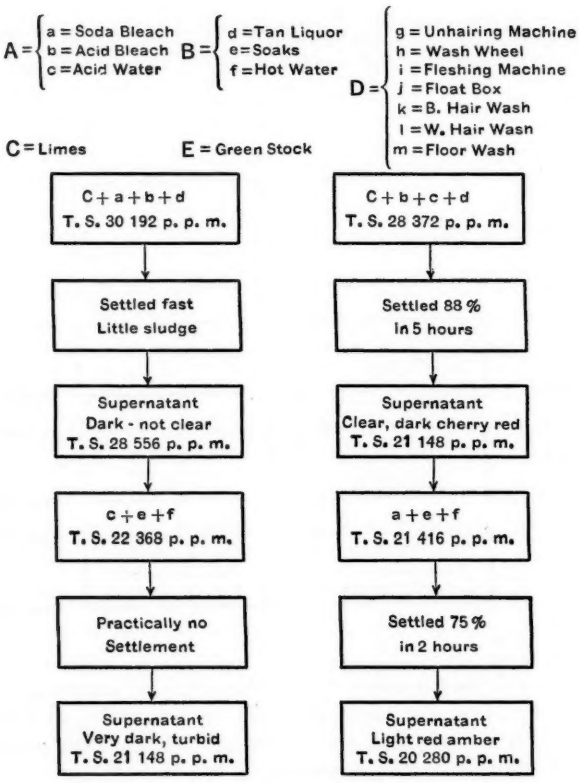
A = a = Soda Bleach
b = Acid Bleach
c = Acid Water
B = d = Tan Liquor
e = Soaks
f = Hot Water
D = g = Unhairing Machine
h = Wash Wheel
i = Fleshing Machine
j = Float Box
k = B. Hair Wash
l = W. Hair Wasn
m = Floor Wash
C = Limes
E = Green Stock
C + a + b + d
T. S. 30 192 p. p. m.
Settled fast
Little sludge
Supernatant
Dark - not clear
T. S. 28 556 p. p. m.
c + e + f
T. S. 22 368 p. p. m.
Practically no
Settlement
Supernatant
Very dark, turbid
T. S. 21 148 p. p. m.
C + b + c + d
T. S. 28 372 p. p. m.
Settled 88 %
in 5 hours
Supernatant
Clear, dark cherry red
T. S. 21 148 p. p. m.
a + e + f
T. S. 21 416 p. p. m.
Settled 75 %
in 2 hours
Supernatant
Light red amber
T. S. 20 280 p. p. m.

FIG. 6.—EXPERIMENT NO. 11. FIG. 7.—EXPERIMENT NO. 12.

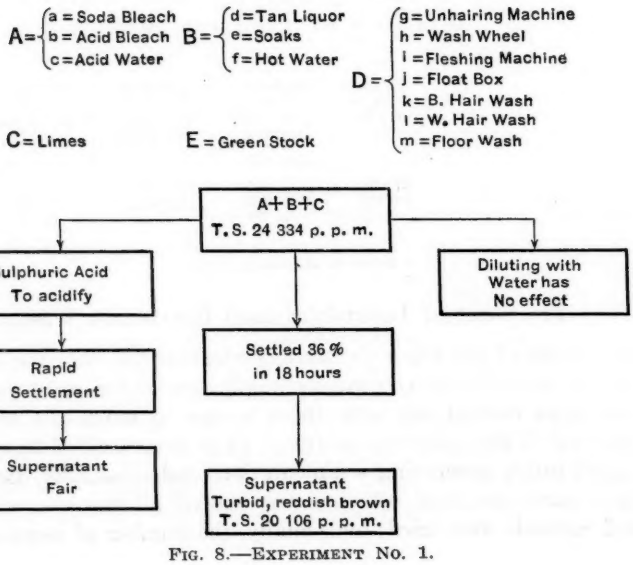

FIG. 8.—EXPERIMENT NO. 1.

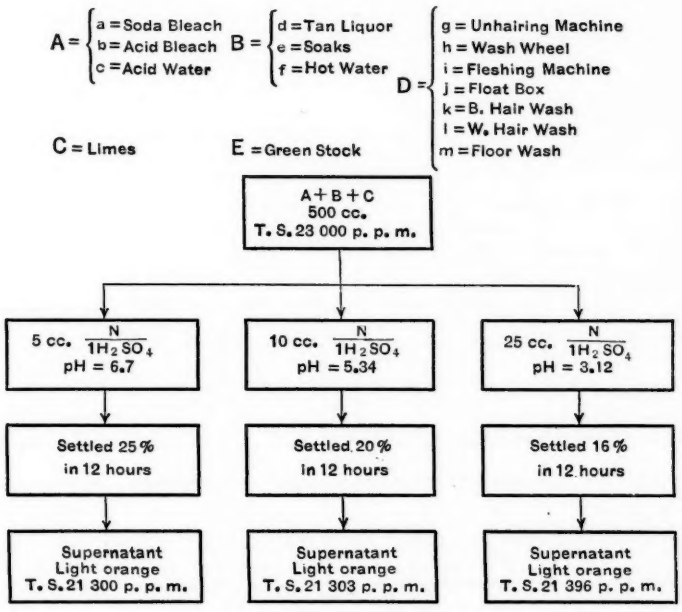
A = a = Soda Bleach
b = Acid Bleach
c = Acid Water
B = d = Tan Liquor
e = Soaks
f = Hot Water
D = g = Unhairing Machine
h = Wash Wheel
i = Fleshing Machine
j = Float Box
k = B. Hair Wash
l = W. Hair Wash
m = Floor Wash
C = Limes
E = Green Stock
A + B + C
500 cc.
T. S. 23 000 p. p. m.
5 cc. N/1H2SO4
pH = 6.7
10 cc. N/1H2SO4
pH = 5.34
25 cc. N/1H2SO4
pH = 3.12
Settled 25 % in 12 hours
Settled 20 % in 12 hours
Settled 16 % in 12 hours
Supernatant Light orange T. S. 21 300 p. p. m.
Supernatant Light orange T. S. 21 303 p. p. m.
Supernatant Light orange T. S. 21 396 p. p. m.

FIG. 9.—EXPERIMENT NO. 9.

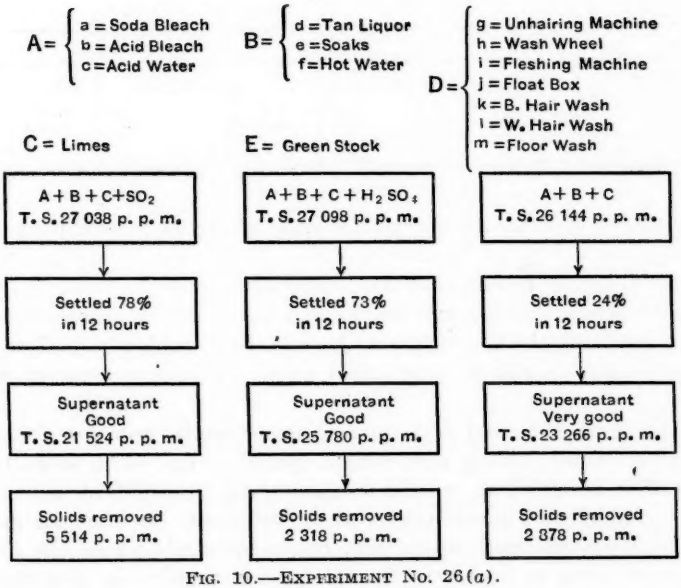
A = a = Soda Bleach
b = Acid Bleach
c = Acid Water
B = d = Tan Liquor
e = Soaks
f = Hot Water
D = g = Unhairing Machine
h = Wash Wheel
i = Fleshing Machine
j = Float Box
k = B. Hair Wash
l = W. Hair Wash
m = Floor Wash
C = Limes
E = Green Stock
A + B + C + SO2
T. S. 27 038 p. p. m.
A + B + C + H2SO4
T. S. 27 098 p. p. m.
A + B + C
T. S. 26 144 p. p. m.
Settled 78% in 12 hours
Settled 73% in 12 hours
Settled 24% in 12 hours
Supernatant Good T. S. 21 524 p. p. m.
Supernatant Good T. S. 25 780 p. p. m.
Supernatant Very good T. S. 23 266 p. p. m.
Solids removed 5 514 p. p. m.
Solids removed 2 318 p. p. m.
Solids removed 2 378 p. p. m.

FIG. 10.—EXPERIMENT NO. 26(*a*).

was reduced to one. At first, it was attempted to produce results by natural interreaction of the wastes; later, studies were made with the addition of the various chemical agents already mentioned.

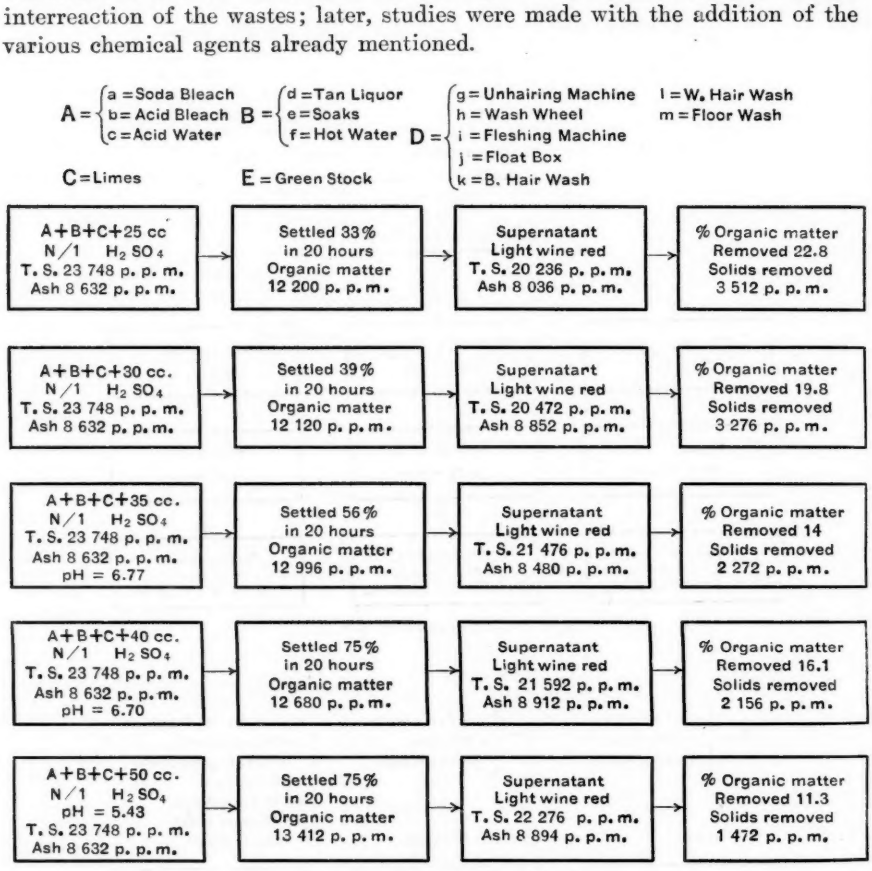
A = a = Soda Bleach, b = Acid Bleach, c = Acid Water
B = d = Tan Liquor, e = Soaks, f = Hot Water
D = g = Unhairing Machine, h = Wash Wheel, i = Fleshing Machine, j = Float Box, k = B. Hair Wash
l = W. Hair Wash
m = Floor Wash
C = Limes
E = Green Stock
A+B+C+25 cc N/1 H2 SO4 T. S. 23 748 p. p. m. Ash 8 632 p. p. m.
Settled 33% in 20 hours Organic matter 12 200 p. p. m.
Supernatant Light wine red T. S. 20 236 p. p. m. Ash 8 036 p. p. m.
% Organic matter Removed 22.8 Solids removed 3 512 p. p. m.
A+B+C+30 cc. N/1 H2 SO4 T. S. 23 748 p. p. m. Ash 8 632 p. p. m.
Settled 39% in 20 hours Organic matter 12 120 p. p. m.
Supernatart Light wine red T. S. 20 472 p. p. m. Ash 8 852 p. p. m.
% Organic matter Removed 19.8 Solids removed 3 276 p. p. m.
A+B+C+35 cc. N/1 H2 SO4 T. S. 23 748 p. p. m. Ash 8 632 p. p. m. pH = 6.77
Settled 56% in 20 hours Organic matter 12 996 p. p. m.
Supernatant Light wine red T. S. 21 476 p. p. m. Ash 8 480 p. p. m.
% Organic matter Removed 14 Solids removed 2 272 p. p. m.
A+B+C+40 cc. N/1 H2 SO4 T. S. 23 748 p. p. m. Ash 8 632 p. p. m. pH = 6.70
Settled 75% in 20 hours Organic matter 12 680 p. p. m.
Supernatant Light wine red T. S. 21 592 p. p. m. Ash 8 912 p. p. m.
% Organic matter Removed 16.1 Solids removed 2 156 p. p. m.
A+B+C+50 cc. N/1 H2 SO4 pH = 5.43 T. S. 23 748 p. p. m. Ash 8 632 p. p. m.
Settled 75% in 20 hours Organic matter 13 412 p. p. m.
Supernatant Light wine red T. S. 22 276 p. p. m. Ash 8 894 p. p. m.
% Organic matter Removed 11.3 Solids removed 1 472 p. p. m.

FIG. 11.—EXPERIMENT NO. 42.

Examples of experiments describing some of the most promising methods of treatment are shown by Experiments Nos. 31 (*a*) and (*b*) (Fig. 12), 32 (*a*) and (*b*) (Fig. 13), 33 and 34 (Fig. 14), 55 (Fig. 15), and 79 (Fig. 16). Several experiments were conducted, treating all the wastes as one single group. This mixture responds to the treatments with alum, lime, acid, etc., and produces a fair effluent. It does not, however, show as good results as some of the other treatments.

In addition to the kind and type of experiments just described, others have been performed dealing with certain special points which seemed supplementary. For instance, in certain tanneries the intermittent wastes might not form as alkaline a conglomerate as at Instanter. In such an event the omission of the soda bleach in the first treatment would lessen the alkalinity and, in some cases, to such an extent as not to require an acid treatment. Again, several experiments were based on the theory that a reaction of the tannin on lime would remove a large amount of solids in the form of an

insoluble calcium tannate. This proved to be the case, but was only practicable when the lime and spent tan were diluted at least once with fairly clean waste, such as the green stock. Treated in this manner the settlement was good and the effluent was fairly clear. A still better procedure included the soda bleach with the lime and tan liquor. When the supernatant from this mixture was added to the remaining wastes, together with alum and sulfuric acid, a very good effluent was produced.

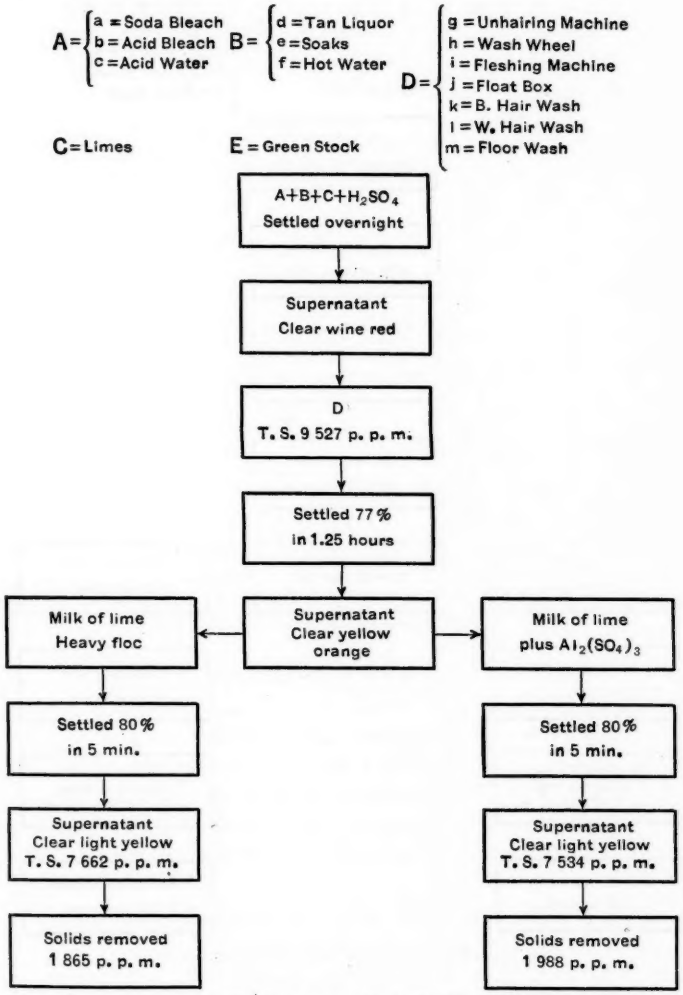
A = a = Soda Bleach, b = Acid Bleach, c = Acid Water
B = d = Tan Liquor, e = Soaks, f = Hot Water
D = g = Unhairing Machine, h = Wash Wheel, i = Fleshing Machine, j = Float Box, k = B. Hair Wash, l = W. Hair Wash, m = Floor Wash
C = Limes
E = Green Stock
A+B+C+H2SO4 Settled overnight
Supernatant Clear wine red
D T. S. 9 527 p. p. m.
Settled 77 % in 1.25 hours
Milk of lime Heavy floc
Supernatant Clear yellow orange
Milk of lime plus Al2(SO4)3
Settled 80 % in 5 min.
Settled 80 % in 5 min.
Supernatant Clear light yellow T. S. 7 662 p. p. m.
Supernatant Clear light yellow T. S. 7 534 p. p. m.
Solids removed 1 865 p. p. m.
Solids removed 1 988 p. p. m.

FIG. 12.—EXPERIMENTS NOS. 31(*a*) AND (*b*).

Several experiments involving the use of lime sludge as a precipitant did not show favorable results.

An experiment permitting the lime and tan liquor to re-act several hours before mixing with the other intermittent waste, did not show favorable

results; that is, there was nothing in the way of improvement to sanction the separate treatment of the tan liquor.

Separate treatment of the bleaches, separate settling of a mixture of the lime and spent tan liquor, and the resulting supernatants mixed with the other wastes, gave very good results.

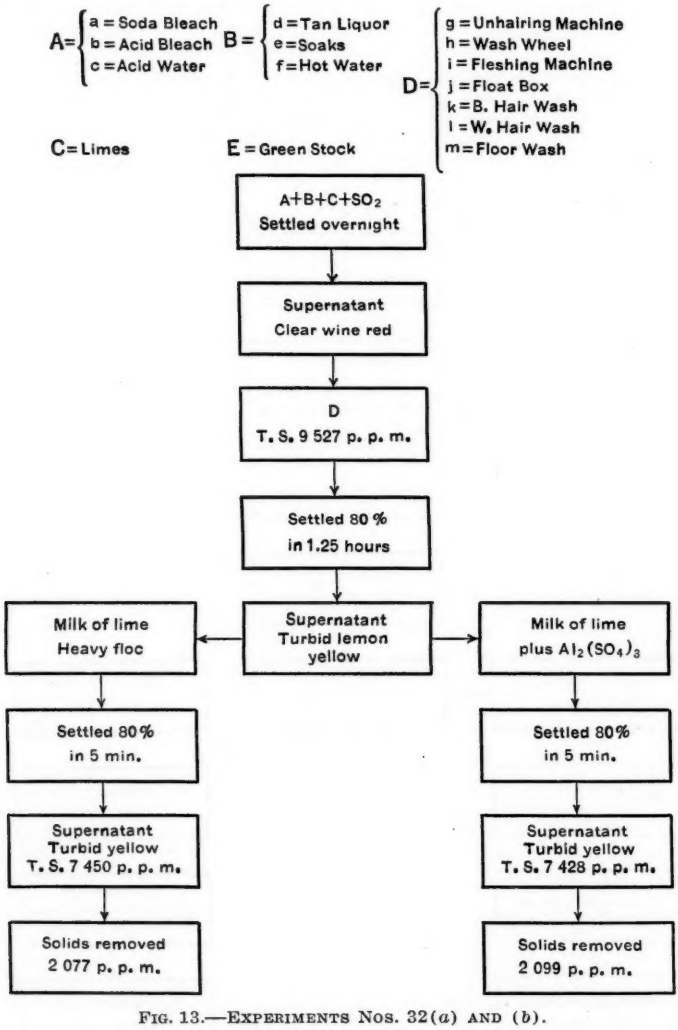
A = a = Soda Bleach, b = Acid Bleach, c = Acid Water
B = d = Tan Liquor, e = Soaks, f = Hot Water
D = g = Unhairing Machine, h = Wash Wheel, i = Fleshing Machine, j = Float Box, k = B. Hair Wash, l = W. Hair Wash, m = Floor Wash
C = Limes
E = Green Stock
A+B+C+SO2 Settled overnight
Supernatant Clear wine red
D T. S. 9 527 p. p. m.
Settled 80 % in 1.25 hours
Milk of lime Heavy floc
Supernatant Turbid lemon yellow
Milk of lime plus Al2(SO4)3
Settled 80% in 5 min.
Settled 80% in 5 min.
Supernatant Turbid yellow T. S. 7 450 p. p. m.
Supernatant Turbid yellow T. S. 7 428 p. p. m.
Solids removed 2 077 p. p. m.
Solids removed 2 099 p. p. m.

FIG. 13.—EXPERIMENTS NOS. 32(*a*) AND (*b*).

Experiment No. 55 (Fig. 15), which is typical of a number of such experiments performed, describes a very good and dependable method of treatment. In this type of treatment the intermittent wastes were mixed together, and the pH of the mixture regulated to about 5.5 by the addition of sulfuric acid. This mixture was then permitted to settle quietly over night as would have been done in actual operation. The next day the supernatant liquid was added to

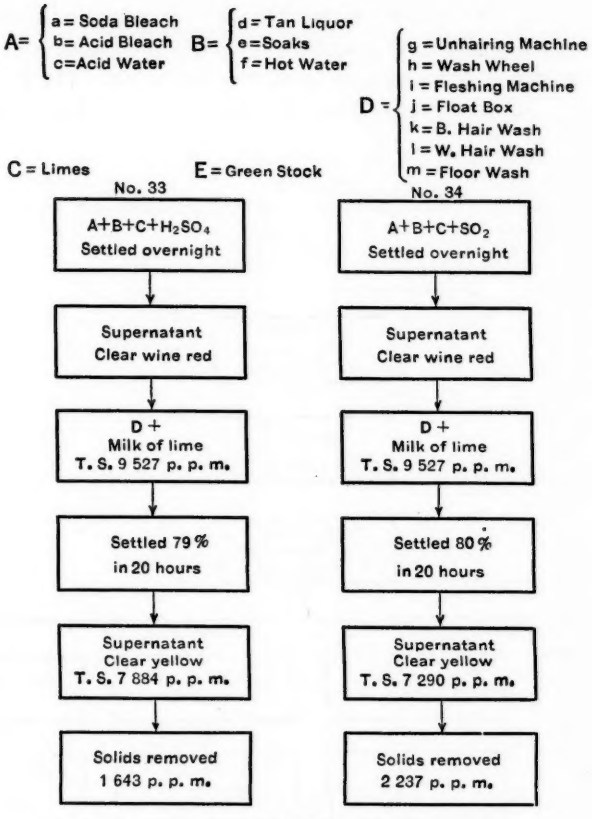

FIG. 14.—EXPERIMENTS NOS. 33 AND 34.

the continuous wastes, except the green stock, together with 20 grains per gal. of aluminum sulfate, and the whole mixed thoroughly. The mixture was then permitted to settle quietly. This particular treatment has been repeated in the laboratory with about twenty-five different samples of the wastes and has been found to give very uniform results. In Table 8 is shown the important determinations made on the final supernatant, which represents the effluent that would be discharged from the tannery after treatment.

TABLE 8.—AVERAGE ANALYSIS OF FINAL SUPERNATANTS, EXPERIMENT NO. 55 (FIG. 15).

	Total solids, in parts per million.	Suspended solids, in parts per million.	5-day oxygen demand, 20° cent., in parts per million.	Color.
Final supernatant,.....	9 335	176	854	1 203
Percentage reduction..	31.1	93.6	57.1	60

Experiment No. 79 (Fig. 16) represents another very good method of treatment, which would probably be useful in tanneries producing larger quantities of spent tan liquor in proportion than that at Instanter. The final effluent of this treatment compares very favorably with that of Experiment No. 55 (Fig. 15).

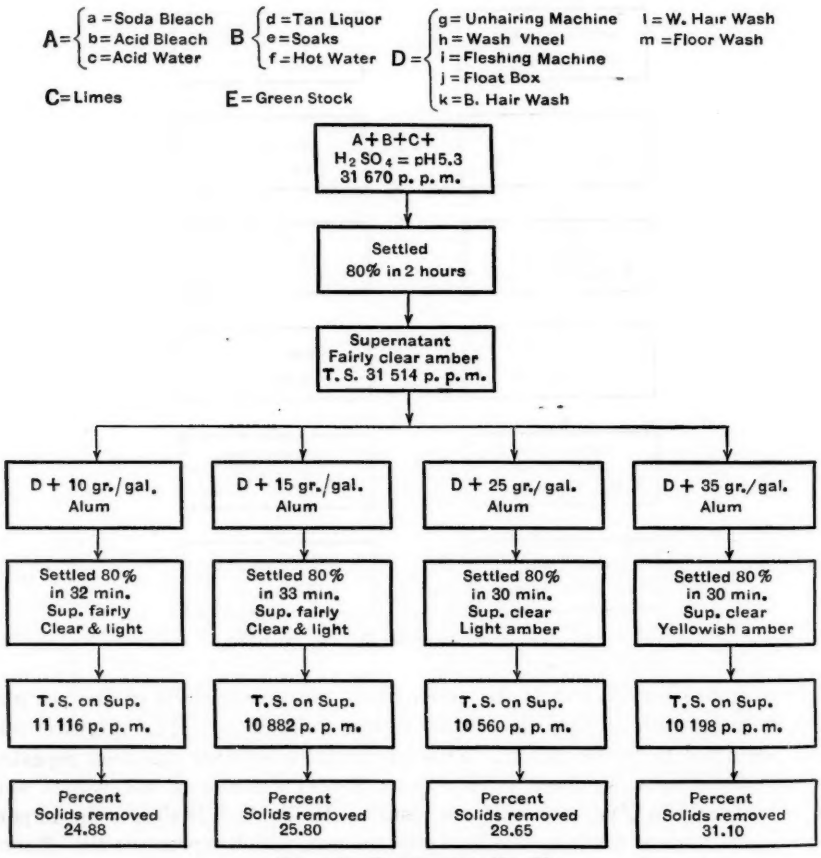
A = a = Soda Bleach, b = Acid Bleach, c = Acid Water
B = d = Tan Liquor, e = Soaks, f = Hot Water
D = g = Unhairing Machine, h = Wash Vheel, i = Fleshing Machine, j = Float Box, k = B. Hair Wash, l = W. Hair Wash, m = Floor Wash
C = Limes
E = Green Stock
A+B+C+ H2SO4 = pH 5.3 31 670 p. p. m.
Settled 80% in 2 hours
Supernatant Fairly clear amber T. S. 31 514 p. p. m.
D + 10 gr./gal. Alum
D + 15 gr./gal. Alum
D + 25 gr./gal. Alum
D + 35 gr./gal. Alum
Settled 80% in 32 min. Sup. fairly Clear & light
Settled 80% in 33 min. Sup. fairly Clear & light
Settled 80% in 30 min. Sup. clear Light amber
Settled 80% in 30 min. Sup. clear Yellowish amber
T. S. on Sup. 11 116 p. p. m.
T. S. on Sup. 10 882 p. p. m.
T. S. on Sup. 10 560 p. p. m.
T. S. on Sup. 10 198 p. p. m.
Percent Solids removed 24.88
Percent Solids removed 25.80
Percent Solids removed 28.65
Percent Solids removed 31.10

FIG. 15.—EXPERIMENT NO. 55.

It is important to note that in treating all the wastes there was strong evidence in favor of the "two-treatment" process. By two treatments is meant separate treatment of the foulest wastes, and then the effluent from this treatment mixed with and treated together with the remaining wastes. Many modifications of the treatments could be arranged, but the basic facts underlying them would be materially the same. The two-treatment process requires a little more equipment, but this is more than offset by the increased efficiency obtained. It is continuous, does not require skilled labor to operate it, and is reliable and inexpensive.

To summarize briefly the laboratory investigations on both the intermittent wastes alone, and then on the intermittent wastes plus the continuous wastes, the following points were noted:

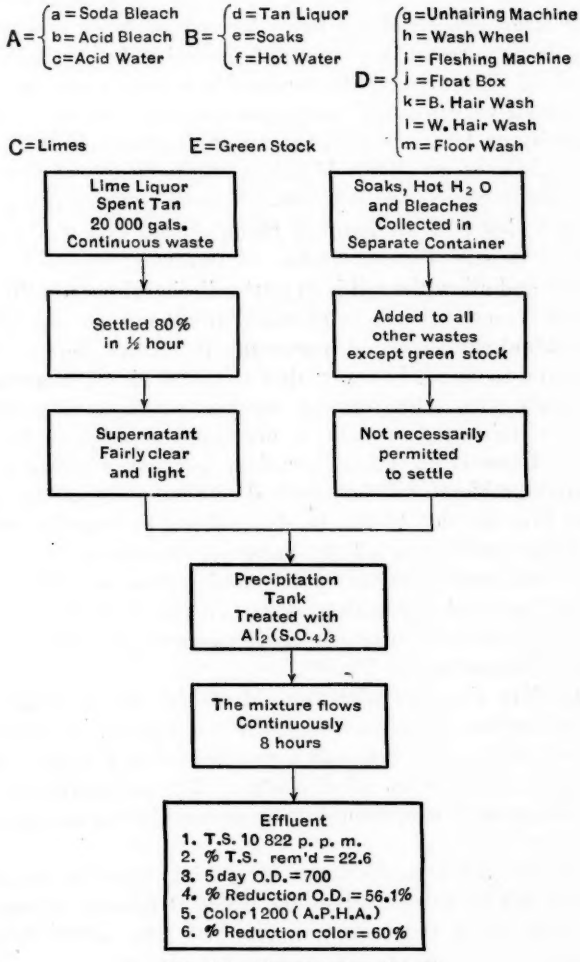
A = a = Soda Bleach, b = Acid Bleach, c = Acid Water
B = d = Tan Liquor, e = Soaks, f = Hot Water
D = g = Unhairing Machine, h = Wash Wheel, i = Fleshing Machine, j = Float Box, k = B. Hair Wash, l = W. Hair Wash, m = Floor Wash
C = Limes
E = Green Stock
Lime Liquor Spent Tan 20 000 gals. Continuous waste
Soaks, Hot H_2O and Bleaches Collected in Separate Container
Settled 80% in ½ hour
Added to all other wastes except green stock
Supernatant Fairly clear and light
Not necessarily permitted to settle
Precipitation Tank Treated with $Al_2(SO_4)_3$
The mixture flows Continuously 8 hours
Effluent
1. T.S. 10 822 p. p. m.
2. % T.S. rem'd = 22.6
3. 5 day O.D. = 700
4. % Reduction O.D. = 56.1%
5. Color 1 200 (A.P.H.A.)
6. % Reduction color = 60%

FIG. 16.—EXPERIMENT NO. 79.

(1) Several mixtures were obtained which gave good results and which did not require chemical treatment.

(2) Chemical treatment, or regulation of the pH of the mixture of intermittent wastes, produced the best improvement.

(3) In treating all the wastes from the tannery it appeared that two treatments were necessary to insure good and uniform results.

Selected Treatments

The most promising of the foregoing experiments were selected for full-scale experimentation and will be described in more detail later. Each of

the treatments selected represents, in the writers' opinions, the best method developed for that particular degree of treatment, based on the laboratory results. The object of the investigation is to solve eventually the problem in enough ways and by enough degrees of treatments to meet practically all the conditions for large and small tanneries located on large and small streams, having plenty or no surrounding land, and operating in expensive or inexpensive environments. The method of procedure in carrying out the investigation, therefore, has been to study the waste treatments in degrees. For instance, for some tanneries on large streams, the direct discharge of their wastes without treatment might be possible. If not, a slight degree of treatment might suffice. This slight degree possibly could be merely the mixing of the combined wastes per day and discharging them, sludge and all, over a selected period of time. A still further degree of treatment would be to mix the combined wastes and allow the solids to settle, discharging only the supernatant liquid. Other refinements, such as chemical treatment, or separate treatment of certain individual wastes, would represent still further degrees of treatment. It should be borne in mind, however, that the methods of treatments referred to have to do only with a preliminary treatment to remove solids and color, etc. Such a preliminary treatment is necessary to prepare the effluent for further degrees of treatment such as aeration, spraying, filtering through sand, etc. These latter problems will be attacked in their order at the proper time, as naturally an investigation of this kind would not be complete without them.

After carefully considering all the various experiments in the laboratory, the following were selected as the most promising to be tried on full scale. They will be referred to henceforth as tests and the description of the experimental plant in which the three tests were operated full scale will be given under Part III, "Experimental Plant".

Test No. I.—Mix (but not with the aid of air) the intermittent waste—limes, soaks, hot waters, bleaches, and spent tan liquor. Discharge the mixture to the stream at an even rate and over a period of 8 hours. Agitate (not by air) during the whole time of discharge. The remainder of the wastes, namely, the continuous wastes, should be discharged in the normal and regular manner.

The object of this test is to discharge everything from the tannery into the river, sludge and all, in order to show the effect of merely mixing the wastes together and discharging them over a period of time rather than releasing them vat by vat.

Test No. IV.—Thoroughly mix the intermittent wastes—limes, soaks, hot waters, bleaches, and spent tan liquor—by plunging up with compressed air. Regulate the pH to about 5.5 by the addition of sulfuric acid. Allow to settle over night. Draw off the supernatant liquid at an even rate and over a period of 8 hours, and discharge directly to the stream; all other wastes to be discharged without treatment. Deposit sludge on sludge beds.

Test No. VII.—Same as Test No. IV up to the point of discharge. Draw off the supernatant liquid and flow by gravity into a precipitation or sedimentation tank at an even rate and over a period of 8 hours. At the same time pump the continuous wastes into the sedimentation tank at an even rate and

over a period of 8 hours. Treat at the entrance of the sedimentation tank with either:* (*a*) Aluminum sulfate; (*b*) milk of lime; or (*c*) aluminum sulfate and milk of lime.

Discharge the effluent from the sedimentation tank to the stream at an even rate and over a period of 8 hours. Sludge to be deposited on sludge beds.

Tests Nos. II, III, and V, not listed here, are modifications of Tests Nos. I, IV, and VII; however, laboratory information, together with the results obtained from selected tests, provide sufficient indication of their behavior on full-scale treatment. For instance, Test No. II is the same as Test No. I, except in the manner of mixing; Test No. II to be mixed by air. Test No. III is the same as Tests Nos. IV or V, except that it is not given an acid treatment. Laboratory results have shown that as such it is impractical. Test No. VI is the same as Test No. VII, except in the manner of mixing.

Part III.—Experimental Plant

The experimental plant is at the Instanter Tannery and has capacity and flexibility enough to handle daily the total wastes from that tannery. The plant comprises sumps, tanks, pumps, engines, and the other necessary equipment for daily continuous operation.

Sumps.—There are two sumps. One, 6 by 10 by 14 ft. deep, is used as an intercepting sump located between the beam house and the main tannery concrete sump. It is so situated that the limes, soaks, and hot water can be separately selected if so desired and retained in this sump. A centrifugal pump is used to deliver the contents from this sump to the mixing sump, which is 32 ft. square and about 6 ft. deep. In this sump, commonly referred to as the 32-ft. sump, those wastes to be treated are delivered by gravity and by pumps and are stored during the day. This sump is equipped with an air-mixing device and a centrifugal pump.

Reaction Tanks.—There are four reaction tanks, each 16 ft. in diameter by 10 ft. deep. They are equipped with air-mixing devices, swivel-jointed draw-off pipes, traps for sludge removal, and other appurtenances.

Sludge Bed.—The sludge bed was not constructed to hold all the sludge produced, only a part of each batch being so placed, the remainder either being run to the creek or held in a lagoon provided for the purpose. The bed is 30 ft. square and is built up of rough gravel, limestone, and cinders, and is well-drained.

Sedimentation Tank.—The sedimentation tank is nothing more or less than a long narrow tank coiled up. Its dimensions, straightened out, would be 4 ft. wide, about 100 ft. long, and 16 ft. deep. It is constructed as a round wooden tank so arranged with two concentric inner tanks as partitions to provide a spiral path of movement for the wastes which enter in the center and move in the spiral path until the periphery of the outer tank is reached, at which point they are drawn off. The round tank is 24 ft. in diameter and 16 ft. deep, and the two inner tanks are 8 ft. and 16 ft. in diameter, the space between the 8-ft. and the 16-ft. tanks being partitioned off. A 2-ft. opening

* Part (*a*) is the recommended method. Part (*b*), or Part (*c*), is to be resorted to only to improve on Part (*a*), if such is possible.

in the side of the 8-ft. tank and extending to the bottom, just ahead of the partition mentioned, permits the wastes to move from the 8-ft. tank into the space between it and the 16-ft. tank. They then proceed around in this 4-ft. channel until the partition is reached. At this point, a 4-ft. opening in the side of the 16-ft. tank, and extending to the bottom, permits the wastes to enter the space between the 16-ft. tank and the outer or 24-ft. tank. A partition between the 24-ft. tank and the 16-ft. tank forces the wastes around the outer channel to the draw-off point located just in front of the partition mentioned. The tank is provided with traps for sludge removal and other appurtenances.

Alum Feed and Mixing Chamber.—The aluminum sulfate solution is mixed in barrels. It is so situated as to feed into the mixing chamber, a 3-ft. square box provided with baffles, at the point where the wastes receive the alum dose. The baffles serve to mix the alum with the wastes.

Operating conditions were necessarily maintained as uniformly as possible in order that the results of the tests would be comparable. The plant was operated, therefore, only when the same kind of wastes were produced, or when the variation from day to day was the least. At the Instanter Tannery this was possible every day, except Saturdays, Sundays, and Mondays. On these days certain intermittent wastes were not produced, but on Tuesdays, Wednesdays, Thursdays, and Fridays, the total amount of the wastes was produced. Other regulations were to limit the time of discharge to some certain period and to maintain an even rate of discharge during this period. These regulations served primarily to aid in comparing the results noted on the receiving body of water. In all the tests, therefore, the same kind of wastes were used, the period of discharge was the same, the rate of discharge over the period was even, and the whole amount of a given waste was used as produced, rather than a proportionate part thereof.

Test No. I

Test No. I, as previously described, was first operated for 5 days in November, 1925, and again for 5 days in July, 1926. Only the intermittent wastes were mixed together, the continuous wastes being discharged in the regular and normal manner. Everything was maintained the same in the two runs, except the method of agitation of the mixture of intermittent wastes during the draw-off, or discharge. In the 1925 test, air was used during the entire 8 hours, whereas in the 1926 test the agitation was accomplished by plunging by hand. The object of the two tests was to obtain the effect on the stream, of discharging the wastes as is, sludge and all. The reason for the different methods of agitating the mixture during the time of discharge was to determine the effect of the air on the mixture.

Operation.—The limes, soaks, and hot waters were pumped daily from the intercepting sump into the 32-ft. sump. Then, the three bleach liquors, namely, soda, acid, and acid water, and, lastly, the spent tan liquor, were delivered to this sump by 4:00 P. M. each day. These wastes, comprising the intermittent wastes, were mixed and pumped to the reaction tanks, leaving the 32-ft. sump empty and ready to accumulate the next day's wastes. The

following day this mixture, while being agitated in the reaction tanks, was withdrawn and discharged at an even rate and over a period of 8 hours.

In the 1925 test an average of 32 000 gal. of the intermittent waste mixture were handled per day and in the 1926 test, 39 856 gal. were produced. The analyses of the mixtures used in the two tests are shown in Table 9.

TABLE 9.—AVERAGE ANALYSIS OF MIXTURES OF INTERMITTENT WASTES.

	NOVEMBER, 1925.*		JULY, 1926.†
	Start of run-off, in parts per million.	End of run-off, in parts per million.	Composite taken over period of discharge, in parts per million.
Specific gravity, 20° cent.	.....	.....	1.0166
Total solids	22 244	22 186	20 552
Soluble solids	19 518	18 612	16 666
Suspended solids	2 726	3 574	3 885
Volatile solids	11 773	9 896	10 461
Fixed solids	10 471	12 290	10 091
Color	8 400	8 600	9 800
pH	6.65	6.70	5.62
5-day oxygen demand, 20° cent.	3 315	1 551	3 876
Alkalinity, CaO	.....	.....	3 388

* Air used for agitation.
† No air used for agitation.

Each analysis in Table 9 is the average of five analyses. The important facts brought out by the analyses of the 1925 test, which were later confirmed by laboratory experimentation, were the decrease of 50% in the 5-day oxygen demand and the increase in color and in suspended solids in the wastes at the end of the run-off period. These differences were due to the aeration effect while agitating the mixture with air.

Test No. IV

Operation.—Test No. IV, as previously described, was conducted during June, July, August, and part of September, 1926. The intermittent wastes were held daily, mixed in the 32-ft. sump, and delivered to the reaction tanks in the same manner as in Test No. I. Here, they were treated with concentrated sulfuric acid until a pH of about 5.5 was reached. The pH was determined colorimetrically. The acid was added by hand from carboys. Usually the correct pH was obtained on the second trial, the whole acid-adding operation requiring less than 30 min. After the acid treatment a sample of the contents of the tanks was preserved in liter-graduated cylinders and permitted to stand quietly over night. In this manner the sludge content could be gauged very precisely the next morning and the quantity of supernatant liquid to be discharged that day could easily be determined. The mixture in the tank was permitted to settle quietly over night. The next morning the supernatant was started discharging at an even rate and for 8 hours thereafter. Composite samples for analysis were taken of the mixture before the acid treatment and of the effluent during the discharge. Whenever the sludge accumulated to the extent that it interfered with the efficiency of the tanks,

it was drawn off, a proportionate part being placed on the sludge beds. The continuous wastes were discharged directly to the creek without treatment in the normal and regular manner.

The average quantity of the daily intermittent waste mixture, based on 16 days' operation, was 37 937 gal., having an average specific gravity of 1.0135 at 20° cent. The average effluent produced per day from this mixture was 27 546 gal., having a specific gravity of 1.0141 at 20° cent. The difference, or 10 391 gal. per day, was retained as sludge, having an average moisture content of 96.78 per cent. In other words, 27.40% by volume of the total daily intermittent waste mixture was retained as sludge. The average sulfuric acid, 66° Baumé, used per day was 206 lb. Table 10 shows the average results of the sixteen analyses of this mixture before treatment and of the effluent as discharged.

TABLE 10.—AVERAGE ANALYSIS OF INTERMITTENT WASTE MIXTURE, BEFORE ACID TREATMENT, AND OF EFFLUENT.

	Before Acid Treatment.		Effluent Discharged to Creek.		Percentage retained as sludge.	Percentage of reduction.
	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.		
Specific gravity, 20° cent.	1.0135	.....	1.0141	.....	.....	.....
Total solids	21 632	6 807	20 581	4 799	30.88	4.86
Soluble solids	19 137	6 140	19 484	4 538	26.10	—1.78*
Suspended solids	2 494	800	1 097	261	67.40	56.10
Volatile solids	9 836	3 177	8 765	2 147	32.48	10.90
Fixed solids	11 729	3 763	11 373	2 652	29.48	3.04
Color	12 132	.....	5 715	.....	.....	52.30
pH	5.81	.....	5.04	.....	.....	.....
5-day oxygen demand, 20° cent	3 773	1 210	3 425	799	33.92	9.22
Total alkalinity, CaO	2 373	762	1 836	428	43.80	24.10

* Increase.

Test No. VII

Just as Test No. VII, as described, was to be started, circumstances over which the Committee had no control caused the Instanter Tannery to cease operations. On September 8, 1926, the Committee was officially notified that hides would cease to be worked in at that tannery on September 11. Efforts were immediately concentrated on Test No. VII, although it was realized that it would probably be impossible to make a thorough study of this test in so short a time. It was possible, however, to obtain a complete and normal daily waste discharge for 9 days, which has indicated, in a general way at least, the character of this test.

Operation.—Test No. VII includes the treatment of all the wastes except the green stock. The intermittent wastes were treated in the same manner as in Test No. IV. The effluent from the intermittent wastes contained in the reaction tanks was discharged into the main trough leading to the sedimentation tank. The continuous wastes were also pumped into this trough. This mixture was discharged into the sedimentation tank at the rate of about

12 000 gal. per hour. Before the mixture entered the sedimentation tank, however, it was passed through a small mixing box at which point a solution of aluminum sulfate was added. The nominal retention period in the sedimentation tank averaged about 2 hours. The effluent was discharged to the creek at the same rate the wastes entered the tank, namely, about 12 000 gal. per hour. The average length of discharge was 8 hours during the working period of the tannery. The contents of the sedimentation tank were permitted to stand quietly during the night. It was found necessary to remove the sludge accumulated in this tank twice during the 14 days it was in operation. The averages of the data collected during 9 days are given in Table 11.

TABLE 11.

Total wastes handled per day.	Quantity.
Continuous and intermittent:	
1. Total, in gallons per day	102 193
Intermittent wastes:	
1. Total in gallons per day	40 406
2. Effluent, in gallons per day	32 700
3. Sludge, " " "	7 706
4. Acid added, in pounds per day	218
Continuous wastes:	
1. Total, in gallons per day	61 787
Combined continuous wastes and effluent from intermittent waste:	
1. Total, in gallons per day	94 487
2. Effluent, in gallons per day	94 487
3. Sludge, " " "	2 980
4. Aluminum sulfate, in pounds per day	250
5. Aluminum sulfate, in grains per gallon	18
Sludge produced:	
1. Sludge from intermittent wastes, in gallons per day	7 706
2. Sludge from sedimentation tank, in gallons per day	2 980
3. Total sludge produced, in gallons per day	10 686

The amount of sludge produced daily in the sedimentation tank was 3.15% by volume based on the daily volume treated in this tank. The moisture content was 97.42 per cent. Considering the total daily volume of wastes, except the green stock, there was an average of 10 686 gal. of sludge produced per day, or 10.45% by volume.

The average results of the nine sets of samples taken during the operation of Test No. VII are presented in Table 12.

In Table 12 the column, "Combined Mixture—No Treatment", is the analysis of the mixture, as produced, of all the intermittent and continuous wastes except the green stock; the column, "Before Alum Treatment", is the analysis of the mixture consisting of the effluent from the intermittent wastes after the acid treatment and the continuous wastes; and the column, "Effluent Discharged to Creek", is the analysis of the effluent from the sedimentation tank. This effluent is produced by treating with aluminum sulfate the mixture, the analysis of which is given in the column, "Before Alum Treatment", and, subsequently, permitting sedimentation to take place in the tank.

The usual preliminary adjustment and regulation of the functioning of the sedimentation tank could not be accomplished before starting this test, owing to the limited time available. Consequently, several alterations, glaringly in need, could not be made. A more efficient method of removing the

sludge would have provided a somewhat longer retention period. Lack of sufficient ground space on which to store the sludge forced the sedimentation tank to operate beyond its point of efficiency, due to sludge piling up inside. This fact partly explains the relatively large amount of suspended solids noted in the effluent. There is no doubt that this tank can be operated much more efficiently when it is properly adjusted.

TABLE 12.—AVERAGE ANALYSIS OF WASTE MIXTURES.

	Combined Mixture.* No Treatment.		Before Alum Treatment.		Effluent Discharged to Creek.		Total amount retained as sludge, percentage.	Amount retained as sludge from sedimentation tank, percentage.	Reduction, percentage.
	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.			
Specific gravity, 20° cent.	1.0071	.....	1.0068	.....	1.0040	.....	.....	.....	.....
Total solids	13 557	11 640	11 687	9 240	6 465	5 100	56.15	44.80	52.25
Soluble solids	10 732	9 224	9 816	7 772	5 614	4 428	52.05	43.00	47.65
Suspended solids	2 825	2 416	1 871	1 468	851	672	72.25	54.20	69.95
Volatile solids	.....	.....	5 928	4 710	3 090	2 440	.....	61.70	.....
Fixed solids	.....	.....	5 759	4 530	3 375	2 660	.....	43.70	.....
Color	3 000	.....	2 747	.....	850	.....	.....	.....	71.70
pH	7.0	.....	5.38	.....	5.69	.....	.....	.....	.....
5-day oxygen demand, 20° cent.	2 000	1 719	1 769	1 399	1 026	808	53.00	42.20	48.60
Alkalinity, CaO	.....	.....	2 290	1 811	1 717	1 364	.....	25.20	.....

* Laboratory mixture.

The point of draw-off, being only 10 in. wide, caused the effluent to be accelerated in that region, resulting in considerable stirring up of the sludge. This trouble can be eliminated by simply making the draw-off trough the width of the channel of the tank. These, and other mechanical features, can easily be adjusted. Color and suspended matter showed 72% and 70% reduction, respectively. The 48.60% reduction in 5-day oxygen demand is also notable because it must be remembered that no special treatment for oxidation, such as aeration, biological filtration, and the like, was made. This reduction in oxygen demand is due primarily to the removal of solids in suspension and in solution. Attention is called to the fact that of the 56.15% removal of solids, 45% was accomplished in the sedimentation tank. This, of course, was by reason of the relatively much larger volume treated in this tank and does not minimize in the least the efficiency of the acid treatment of the intermittent wastes because the efficacy of both treatments has already been ably demonstrated by these tests. The removal of approximately five-sevenths of the total suspended matter and two-thirds of the total oxygen demand was accomplished in the sedimentation tank.

The success of this test is evident from the actual facts noted. The production of a suitable effluent has consumed practically the entire attention of the investigation to date. A detailed study of the sludge problem will be taken up as soon as the experimental plant has been transferred to another tannery.

Although the abandonment of Instanter Tannery cut short further experiments, the information derived from the tests is valuable.

Summary of Full-Scale Operations

1.—Test No. I, which amounts to mixing certain wastes, and prolonging their discharge, undoubtedly could be used to advantage by some tanneries.

2.—The use of air for mixing produced a 50% reduction in the oxygen demand of the untreated intermittent wastes.

3.—Test No. IV proved to be a dependable method of producing sedimentation of the intermittent waste mixture at little cost and little labor.

4.—Test No. VII, although not operated long enough to prove its ultimate efficiency, clearly indicated an effective treatment.

Part IV.—River Studies

River studies were made to determine the effect of the various treatments described in the preceding section, on the East Branch of the Clarion River, immediately below the point of discharge of the wastes, and for a distance down stream of about 12 miles. The river studies were carried on only during the summers of 1925 and 1926. Most of the river work of the summer of 1925 consisted in determining its condition during the regular and normal daily operations of the tannery. The river investigation during the summer of 1926 has consisted only of the study of Tests Nos. I, IV, and VII.

The samples of the river water obtained for analysis were taken from carefully selected stations located at definite points along the river over the 12-mile stretch.

Station 1 was immediately below the point of discharge of the tannery waste, but far enough down stream to insure thorough mixing of the water and waste.

Station 2 was 1½ miles below Station 1, and just above the junction of Straight Creek with the Clarion.

Station 3 was 3 miles below Station 1 and 1½ miles below Station 2, just above the junction of Swamp Creek with the Clarion.

Station 4 was 5.6 miles below Station 1 and 2.6 miles below Station 3, about ½ mile above the junction of Crooked Creek with the Clarion.

Station 5 was about 12 miles below Station 1 and 6 miles below Station 4.

The river was accurately gauged at each station so that the discharge, area, and velocity could be determined whenever desired. The time of flow between stations was also determined a number of times.

During the sampling of the river no sludge or other extraneous matter likely to mask the effect of the treated wastes was permitted to discharge into it. Samples of the river water were collected at each station during 24-hour periods—one sample taken each hour during the period. The dissolved oxygen, temperature, pH, and turbidity were determined in the field, and the total solids, soluble solids, suspended solids, color, oxygen demand, etc., in

sludge would have provided a somewhat longer retention period. Lack of sufficient ground space on which to store the sludge forced the sedimentation tank to operate beyond its point of efficiency, due to sludge piling up inside. This fact partly explains the relatively large amount of suspended solids noted in the effluent. There is no doubt that this tank can be operated much more efficiently when it is properly adjusted.

TABLE 12.—Average Analysis of Waste Mixtures.

	Combined Mixture.* No Treatment.		Before Alum Treatment.		Effluent Discharged to Creek.		Total amount retained as sludge, percentage.	Amount retained as sludge from sedimentation tank, percentage.	Reduction, percentage.
	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.	Parts per million.	Pounds per day.			
Specific gravity, 20° cent.	1.0071	.....	1.0068	.....	1.0040	.....	.....	.....	.....
Total solids	13 557	11 640	11 687	9 240	6 465	5 100	56.15	44.80	52.25
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Fixed solids	.....	.....	5 759	4 530	3 375	2 660	.....	43.70	.....
Color	3 000	.....	2 747	.....	850	.....	.....	.....	71.70
pH	7.0	.....	5.38	.....	5.69	.....	.....	.....	.....
5-day oxygen demand, 20° cent.	2 000	1 719	1 769	1 399	1 026	808	53.00	42.20	48.60
Alkalinity, CaO	.....	.....	2 290	1 811	1 717	1 364	.....	25.20	.....

* Laboratory mixture.

The point of draw-off, being only 10 in. wide, caused the effluent to be accelerated in that region, resulting in considerable stirring up of the sludge. This trouble can be eliminated by simply making the draw-off trough the width of the channel of the tank. These, and other mechanical features, can easily be adjusted. Color and suspended matter showed 72% and 70% reduction, respectively. The 48.60% reduction in 5-day oxygen demand is also notable because it must be remembered that no special treatment for oxidation, such as aeration, biological filtration, and the like, was made. This reduction in oxygen demand is due primarily to the removal of solids in suspension and in solution. Attention is called to the fact that of the 56.15% removal of solids, 45% was accomplished in the sedimentation tank. This, of course, was by reason of the relatively much larger volume treated in this tank and does not minimize in the least the efficiency of the acid treatment of the intermittent wastes because the efficacy of both treatments has already been ably demonstrated by these tests. The removal of approximately five-sevenths of the total suspended matter and two-thirds of the total oxygen demand was accomplished in the sedimentation tank.

The success of this test is evident from the actual facts noted. The production of a suitable effluent has consumed practically the entire attention of the investigation to date. A detailed study of the sludge problem will be taken up as soon as the experimental plant has been transferred to another tannery.

Although the abandonment of Instanter Tannery cut short further experiments, the information derived from the tests is valuable.

Summary of Full-Scale Operations

1.—Test No. I, which amounts to mixing certain wastes, and prolonging their discharge, undoubtedly could be used to advantage by some tanneries.

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Part IV.—River Studies

River studies were made to determine the effect of the various treatments described in the preceding section, on the East Branch of the Clarion River, immediately below the point of discharge of the wastes, and for a distance down stream of about 12 miles. The river studies were carried on only during the summers of 1925 and 1926. Most of the river work of the summer of 1925 consisted in determining its condition during the regular and normal daily operations of the tannery. The river investigation during the summer of 1926 has consisted only of the study of Tests Nos. I, IV, and VII.

The samples of the river water obtained for analysis were taken from carefully selected stations located at definite points along the river over the 12-mile stretch.

Station 1 was immediately below the point of discharge of the tannery waste, but far enough down stream to insure thorough mixing of the water and waste.

Station 2 was 1½ miles below Station 1, and just above the junction of Straight Creek with the Clarion.

Station 3 was 3 miles below Station 1 and 1½ miles below Station 2, just above the junction of Swamp Creek with the Clarion.

Station 4 was 5.6 miles below Station 1 and 2.6 miles below Station 3, about ½ mile above the junction of Crooked Creek with the Clarion.

Station 5 was about 12 miles below Station 1 and 6 miles below Station 4.

The river was accurately gauged at each station so that the discharge, area, and velocity could be determined whenever desired. The time of flow between stations was also determined a number of times.

During the sampling of the river no sludge or other extraneous matter likely to mask the effect of the treated wastes was permitted to discharge into it. Samples of the river water were collected at each station during 24-hour periods—one sample taken each hour during the period. The dissolved oxygen, temperature, pH, and turbidity were determined in the field, and the total solids, soluble solids, suspended solids, color, oxygen demand, etc., in

the laboratory, all results being calculated to standard discharge conditions. The standard drought flow discharges for the sampling stations are as follows:

Station 1	5.0	sec-ft.
" 2	5.0	" "
" 3	7.9	" "
" 4	10.2	" "
" 5	13.9	" "

Although figures are submitted in this paper, the writers feel that final conclusions cannot be given at this time. It is known that a number of foreign factors have upset, to some extent, the results, and before these factors and their influence have been fully analyzed, definite statements cannot be made.

Preliminary Stream Study, Summer, 1925

In the normal and regular daily operation of the tannery the wastes are discharged, sludge and all. The practice is to remove the plug in the vats and permit the wastes to discharge directly to the creek. Most of the heavy lime sludge deposited in the bottom of the lime vats is shoveled out in a pile, but does not find its way to the stream. The continuous wastes are pumped to the stream directly. The only waste from the Instanter Tannery which does not regularly discharge into the creek is the spent tan liquor. This waste is pumped up into earthen lagoons, situated above the tannery, in which the liquor is permitted to settle and otherwise to trickle over and filter through the sides, and down the hill. In this manner considerable oxidation occurs before the waste finally reaches the river.

The average analysis of the 24-hourly samples by stations is given in Table 13. The column, "1", is the analysis of the water at Station 1, and is comparable in all respects with the analyses of the other stations given in the table, except that in this case the spent tan liquor was discharged directly to the creek, whereas in the others it was pumped into the lagoons at the tannery. Only one station was sampled with the tan liquor discharged direct, because the officials of a large paper plant at Johnsonburg, 12 miles below, seriously objected to its presence in the water.

TABLE 13.—Analysis by Stations. Tannery Producing Normal and Regular Daily Wastes.

	Station.					
	1.*	1.	2.	3.	4.	5.
Total solids, in parts per million	448	417.5	339	427.5	893.0	131
Dissolved oxygen, in parts per million	2.44	†	4.2	5.4	6.9	8.21
Dissolved oxygen, percentage saturated	33.13	†	48.4	58.9	75.0	88.0
5-day oxygen demand, 20° cent	40.50	28.1	6.4	11.5	3.3	0.89
Color	1 368	219.5	144	394.5	245.5	17.4
Turbidity	453	65.9	14.4	54.3	21.8	5.70
pH	6.10	6.6	6.8	6.9	6.9	6.95
Temperature, in degrees, centigrade	18.21	16.7	21.	16.8	17.4	17.91
NaCl, in parts per million	.....	62.3	112	69.3	55.6	19.6

* Spent tan liquor discharged direct.

† No samples.

Test No. I, November, 1925, and July, 1926

In this test each station except Station 3 was sampled twice, once in November and once in July. The average analysis for each station is given in Table 14.

TABLE 14.—Average Analysis by Stations, Test No. I.

	Stations.						
	1.	2.	4.	5.	Instanter Creek.	Seven-Mile Run.	Straight Creek.
Total solids, in parts per million	483	1 071	792	600	66.0	69.0	50.0
Soluble solids, in parts per million	371	851	584	482	....	....	....
Suspended solids, in parts per million	112	220	208	198	....	....	....
Dissolved oxygen, in parts per million	4.99	7.59	6.86	8.85	8.4	7.7	7.4
Percentage of saturation, dissolved oxygen	45.78	60.32	61.57	80.66	91.3	84.4	86.2
5-day oxygen demand, 20° cent	47.69	71.33	31.95	12.97	0.17	0.33	0.73
Color	654	457	340	195	12.5	18.8	10.0
Turbidity	116	155.4	111	26	0	0	0
pH	6.5	6.44	6.67	6.8	6.0	5.4	6.8
Temperature, in degrees centigrade	10.06	11.24	13.43	12.35	17.3	17.3	20.3

The analyses of Instanter Creek, Seven-Mile Run, and Straight Creek are given in Table 14 to show the character of the water of these tributaries. Instanter Creek and Seven-Mile Run are the two little creeks which come together just below the tannery, to form the East Branch of the Clarion River. Station 1, as stated, was just below the junction of these two creeks. The samples obtained for analysis from these creeks were taken immediately above the tannery on Seven-Mile Run, and just above the junction point on Instanter Creek. Straight Creek flows into the Clarion at Straight, Pa., 1½ miles below Station 1. A practice was made throughout the investigation of analyzing the waters of these creeks frequently in order to detect outside sources of pollution.

As in this test everything produced as waste at the tannery is discharged to the river, except the lime sludge, it should constitute the worst condition. Consequently, the removal of any suspended matter from the river should lessen the load on the stream to a noticeable degree. In Test No. IV, in which the intermittent wastes were treated with sulfuric acid, permitted to settle, and only the supernatant liquid permitted to flow to the creek, the retention of the settled material should be noticeable by a general improvement in the condition of the river.

Test No. IV, Summer, 1926

The stations were sampled once each in June, July, and August, during the operation of Test No. IV. The average analysis for each station, of the three times sampled, is given in Table 15.

There is an improvement noted in these analyses over those made for Test No. I. The condition noted farther down stream, however, is not as

good as was expected. It is difficult to ascertain at present just how much influence the wood distillation plant, situated on Straight Creek, several hundred feet from its mouth, had on the results below Station 2. All the samples below this point contained a strong odor of wood creosote, and Straight Creek showed a much higher oxygen demand than was noted before. The wood distillates are composed of a large proportion of unsaturated compounds and have a very high oxygen consumed value. The slight yellow color imparted to the water from the wood oils and tars seems to predominate for great distances, only dilution tending to diminish it.

TABLE 15.—AVERAGE ANALYSIS BY STATIONS, TEST NO. IV.

	Stations.				
	1.	2.	3.	4.	5.
Total solids, in parts per million	378	448	259	242	561
Soluble solids, in parts per million	331	414	220	206	527
Suspended solids, in parts per million	47	34	39	36.1	34
Dissolved oxygen, in parts per million	4.71	4.74	5.44	5.91	7.26
Percentage of saturation, dissolved oxygen	48.42	50.66	56.93	65.91	77.94
5-day oxygen demand, 20° cent	33.07	17.30	18.22	7.65	5.60
Color	269	262	247	277	198
Turbidity	43.7	52.6	24.4	18.05	26.2
pH	6.06	6.59	6.76	6.77	6.73
Temperature, in degrees centigrade	17.08	18.23	17.18	18.85	17.03

The dissolved oxygen and the percentage saturation of dissolved oxygen increased steadily with distance from the tannery. The turbidity, color, and oxygen demand decreased steadily. Most of the improvement seemed to have been accomplished in the first three miles below the tannery. From that point the change was very gradual.

The analysis was not the only means of detecting the improvement in the river condition. The removal of the sludge, of course, disposed of considerable suspended solids, the effect of which was very marked at Station 1. Throughout this test the water was clear enough to see the bottom plainly. At no time during the summer of 1925 and during the operation of Test No. I in July, 1926, was this possible. The quantity of small fish in evidence near Station 1 during this test was considerably greater than before, and the jet black appearance of the river bed seemed to have been washed to a more natural color. Thus, not only by analysis, but also by observation, was an improvement noticed at Station 1.

Test No. VII, September, 1926

The effluent from the tannery during this test hardly discolored the water at Station 1. The color at this station at the time of sampling was 33, barely noticeable in an ordinary household tumbler. This represented more than a 50% reduction in the color at this station.

The average analyses of the stations made during Test No. VII are given in Table 16.

TABLE 16.—AVERAGE ANALYSIS BY STATIONS, TEST NO. VII

	Stations.							
	1.	2.	3.	4.	5.	Instanter Creek.	Seven-Mile Run.	Straight Creek.
Total solids, in parts per million....	662.01	521.85	523.6	461.98	244.4	44	53	34
Soluble solids, in parts per million.	587.61	464.13	465.1	361.48	227.6	42.4	51.8	29.2
Supended solids, in parts per million.....	74.4	57.72	58.5	100.5	16.8	1.6	1.2	4.8
Dissolved oxygen, in parts per million.....	8.83	7.90	7.84	8.44	8.09	9.76	9.94	9.24
Percentage of saturation, dissolved oxygen.....	86.21	78.95	79.09	84.19	84.13	98.57	97.81	90.67
5-day oxygen demand, 20° cent.....	19.05	9.20	32.19	17.54	1.79	0.38	0.05	1.21
Color.....	293	245	385	224	188	30	45	45
Turbidity.....	43.87	31.12	43.89	32.30	20.2	4	4.5	
pH	6	6.3	6.1	6.3	6.3	5.6	5.7	6.3
Temperature, in degrees centigrade	12.8	14.09	14.24	13.89	15.6	14.15	12.5	12.9

Conclusions

In order to compare broadly the results obtained from the river during the various tests, Table 17 is presented.

TABLE 17.

Station.	Test.	Saturation dissolved oxygen, percentage.	5-day oxygen demand, 20° cent., parts per million.	Color.	Turbidity.
1.....	No. I	45.78	47.69	654	116
	" IV	48.42	33.07	269	44
	" VII	86.21	19.05	293†	44
2.....	" I	60.32	*71.33	457	*155
	" IV	50.66†	17.30	262	* 53
	" VII	*78.95	9.20	245	31
3.....	" I	56.93	*18.22	247	24.4
	" IV	*79.09	*32.19	*385†	* 44†
	" VII	61.57	31.95	340	111
4.....	" I	65.91	7.65	*277	18
	" IV	*84.19	*17.54	224	* 32
	" VII	80.66	12.97	195	26
5.....	" I	77.94†	5.60	198	* 26
	" IV	*84.13	1.79	188	20.2

* These figures are out of line when comparing the test results by stations.
† These figures are out of line when comparing the results of tests at each station.

In Table 17, Tests Nos. IV and VII show a decided reduction at each station in oxygen demand, assuming that Test No. I represents the worst condition. Paralleling this reduction throughout is the increase in the percentage saturation of dissolved oxygen. The color likewise shows improvement. It may be, however, that the colors noted in the lower stations are normal, that is, that the colors in the upper stations are not very persistent and are easily and quickly diffused or lost. The average color of the waters at Station 4 and at Station 5 at the time of sampling was about 25 and 12, respectively.

It is practically impossible to make conclusive statements about the river work. The importance, however, of the river studies cannot be over-estimated. The results thus far obtained demonstrate that it was possible to observe and show by analyses that Test No. IV was an improvement on Test No. I, and that Test No. VII was an improvement on Test No. IV. The fact that it was possible to bring out these differences in the water is a most important feature. It is hoped by future work to be able to calculate, or to forecast, the effect of any concentration and volume of tannery waste on any river.

General Summary

1.—Chemical analysis of the individual wastes revealed the fact that the intermittent wastes contributed about 75% of the total putrescence. Their volume was only 28% by volume of the total daily wastes.

2.—The laboratory experiments indicated that chemical treatment was necessary for dependable results.

3.—Several different types of treatment were developed in the laboratory.

4.—Two full-scale tests accomplished satisfactory removal of suspended matter in the wastes treated, one treating the intermittent wastes only, and the other treating all the wastes, except the green stock.

5.—The river studies proved to be important. The analyses of the river water showed that about 50% of the improvement is accomplished in the first 3 miles of the 12-mile stretch immediately below the tannery.

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EARTH WORK BY THE HYDRAULIC METHOD*

By Roy E. Miller,† M. Am. Soc. C. E.

This subject will be discussed from the standpoint of the contracting engineer, with particular reference to the aid that can be derived in making estimates of the cost of earth work from a series of laboratory tests of samples of the earth to be moved.

When a project is under consideration, one of the vital factors to be given attention by all concerned is, "What will the work cost?" The point of view of each interested party depends on his personal interest in the work. When the project has progressed to the stage where it comes to the attention of the contracting engineer, the estimate of the cost is the all-absorbing factor. The contracting engineer goes a step farther than the project engineer in that he is obliged to make good his idea of cost by personal performance and financial guaranties. This means that every assistance that he can call on makes for security in his business, and helps to stabilize engineering and construction.

The character of the soil is the controlling factor in the cost of the work where material is to be excavated and placed by hydraulic dredging or by sluicing. Throughout this paper when reference is made to an hydraulic dredge, it should be understood to mean a pipe-line dredge of the radial type, that is, one swinging from a "spud" anchorage, and with a cutter for an agitator, which machine has come to be practically a standard tool for contractors' use in the United States. Sluicing, likewise, is meant to cover excavation by giants or monitors, and delivery through flumes by gravity or by booster pumps through pipes to the place of deposit.

The character of the material determines the difficulty or ease of digging or excavating, of pumping, and of retaining within the fill area. The writer's experience includes fills behind bulkheads to reclaim tide lands and marsh lands, to grade streets, platted city property, and mill and factory sites, to

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build dams for storage and river diversion for irrigation projects and city water supply, to replace railway trestles and bridges, and to permit the construction of harbor works, piers, docks, elevators, and warehouses.

The methods of estimating the cost of such work have been unsatisfactory on account of the many unknown and incomputable elements as compared with the degree of accuracy in estimating other engineering projects involving dry earth work, concrete, steel, etc.

In December, 1921, the Puget Sound Bridge and Dredging Company took a contract for building a storage dam on the Big Wichita River, about 50 miles from Wichita Falls, Tex. This dam was to be an hydraulic fill, 100 ft. high, with a reservoir capacity of approximately 550 000 acre-ft. In determining whether to take this work the Company was faced immediately with two important problems. The first was whether the borrow field contained the proper materials for making a satisfactory hydraulic fill and dam, should a large hydraulic dredge be used as was intended. Samples of the material were obtained, and Mr. I. F. Laucks, a chemist and assayer of Seattle, Wash., was called in to assist in analyzing the materials in order to arrive at an answer to this problem. The second—the cost of the work—could only be determined by rough comparison with results achieved elsewhere and of necessity was based wholly on personal knowledge and experience of the probable monthly output of the dredge. This, in turn, largely determined the unit cost of material handled, as the cost of operating the dredge was comparatively easy to estimate.

Dams have been built by sluicing methods for many years, and it was the experience gained therein that gave the Company the added confidence necessary to undertake to build the first dam with a dredge. Believing that a new field was being entered, wherein hydraulic dredging would find a place in the construction of earth dams, and knowing the difficulty of estimating work of this kind, and, further, finding so much aid from Mr. Lauck's laboratory work, the next logical step was to define some method of examining samples of earth taken from a proposed borrow field, which would assist in estimating the cost of work, particularly the probable output of a given dredge and the action of that material in a dam or fill.

Accordingly, it was decided to learn whether it would be possible to formulate a set of standard tests to be used as a basis for comparing specimens, as has been done with cement, steel, and other materials. It was hoped that a basis of comparison of earth samples could be established which would furnish a more or less correct idea of the probable action of the earth and of the performance of the machinery to be placed on the work.

It was recognized that any basis of predicting the behavior of a new earth or soil for dredging work would have to be comparative. There are no methods of testing soil to determine the cost of dredging. However, by determining the properties of various kinds of earths of which the characteristics and cost factors were known, and comparing them with the physical properties of a new earth it was believed that such comparisons could be used as the basis for predicting the cost of working the new earth.

After considerable preliminary work had been done, it appeared that the most important element in a soil, as affecting its ease or difficulty of working, was the clay content and the special properties of the clay. Clay is the binding element which makes soils stiff, hard to dig, hard to disintegrate, sticky, etc. There are, of course, certain earths which might be cemented by calcareous or siliceous binder, but these are not nearly so common as clay binders. Looking at this problem in another way, a pure sand or gravel, or even a pure silt, presents no particular difficulties as far as digging is concerned. It is only when the sand or silt is held together by some binding agent that difficulty begins, and the most common binding agent is clay.

Clay is merely an empirical term for particles of a certain range in size and does not signify any pure substance with certain specific properties. In other words, clays will vary among themselves very greatly, depending principally on their fineness.

In this paper all particles are considered as clay which, when suspended under proper conditions in a considerable volume of water, refuse to settle in 24 hours, through a depth of 15 cm. This gives particles of a maximum size of 0.001 mm. Some authorities have defined clay as all particles less than 0.001 mm. in size; others include as clay all material less than 0.01 mm. Taking the definition of clay and considering the maximum size as 0.001 mm., it will be noted that there are all sizes from this upper size down to particles infinitely fine; that there is as much variation between the smallest and largest clay particles as there is between a fine silt and a large cobblestone. It is generally agreed that the special properties of a clay, as, for instance, its plasticity, stickiness, binding properties, etc., depend on the fact that it is a finely subdivided material, and, moreover, that these properties vary with the fineness, and shape. The surface area of the clay particle, or rather the ratio of the surface area to the mass, seems to be the important factor, and this ratio increases enormously with the fineness, and varies with the shape, of particles.

The following tests which are proposed for each sample of earth, are not meant to be necessarily final, as it may be found that certain tests are not of much value and may be dropped, or that others are needed. However, it is believed that the following tests will be fairly comprehensive.

1.—The Percentage of Water Required to Bring the Earth to the Plastic Stage.—Instead of leaving this determination to the judgment of the laboratory operator, it is proposed to measure the penetration of a weighted needle into the mass of clay, which is fairly definite and can be checked quite closely. In experimental work the ordinary asphalt penetrameter is used. A penetration of 80° in 15 sec., the needle having no additional weights added, was adopted as standard.

2.—The Percentage of Water in the Earth in Its Natural Condition.—The test is to find plasticity of the earth in its natural condition as determined by the distance to which the needle will penetrate under the same conditions as in Test 1.

Tests 1 and 2, therefore, will indicate the stage of the earth in its natural condition as compared with the plastic stage. For example, if an earth in its

natural state has a penetration of the needle of 20°, it is certain that it will require much more effort to dig than it would if it were at the plastic state. So another earth having a penetration of 40° would be easier to dig than one with a penetration of 20 degrees. If an earth required 20% of water to bring it to the plastic stage, that is, with a penetration of 80°, it would be found that the earth in its natural state, with a penetration of 20°, would contain perhaps 10% of water. If, on the other hand, a natural earth had a penetration of 140° one would expect it to be easier to dig, as far as the power requirements are concerned, but it would be very sticky and might cause trouble by adhering to the cutter, etc.

3.—Determination of the Percentage of Clay, Silt, and Sand.—Clay is considered to be all those particles that fail to settle through 15 cm. of water in 24 hours. This gives the maximum size of 0.001 mm.

Silt is considered to be all particles which settle through 15 cm., in less than 24 hours, but fail to settle in 1 min. This gives a range of particles from 0.001 mm. as the finest to about 0.01 mm. as the maximum.

Sand is all particles coarser than the silt, subject, of course, to the distinction between sand and gravel.

Wherever the sand content of an earth is considerable, a screen test should be made on the sand. In the preliminary work a number of samples were graded into six fractions finer than sand. All the fractions were obtained by water settling through various periods and were then examined under the microscope and measured, but it was not considered necessary to do this in routine testing.

4.—Absorption of Water Vapor and a Dye.—This test is designed to distinguish clays of varying degrees of fineness by taking advantage of their ability to obsorb either water vapor or a dye. The dye used has been methyline blue. The finer the clay, the more of either is absorbed. It has been found that silt and sand absorb very little, so that the main absorption is due to the clay portion. This test is made on a dry earth.

5.—Rise of Water by Capillarity.—This test is made by filling a glass tube to a depth of 20 cm. with a dry earth packed by tamping until no further subsidence occurs. The lower end is closed with loose cloth and immersed in water. The rise of the water through the earth in the tube is measured from time to time; the time required for the water to rise 20 cm. is taken as the unit. Curves of time and height of rise for various earths showed a marked difference. This is largely a comparative test, but it is believed that further experiments with it will enable it to be made of considerable value in classifying an unknown earth.

6.—Percentage of Waters Absorbed by Capillary Action.—This is determined by pushing out the wet column of earth in Test 5 after the water has risen to the top and determining the percentage of water content. This is a comparative test, and is very important in determining certain actions of the material.

7.—Percentage of Water When Saturated.—This is determined by allowing some of the dry earth to absorb all the water that it will. The percentage of water is then determined.

Tests 5, 6, and 7 would also have an immediate practical bearing where the bank of dry earth to be dug will be submerged by the dredging pond. In such case, it is important to know how this bank of dry earth will absorb water by capillary action and how it will act after the water has been absorbed.

8.—Shrinkage in Volume Between the Saturated Condition and the Dry Condition.—This is determined by drying the saturated earth of Test 7 and measuring the contraction in volume.

9.—Slaking or Disintegrating Tendency of Natural Earth.—A piece of natural earth is immersed in water and the time required for the earth to fall to pieces is measured. Great differences have been found in samples tested. This is intended as a comparative test, and also as of immediate practical importance in considering earths that are to be passed through pipe lines.

10.—Water Absorbed by an Earth in Its Natural State and the Change in Plasticity Due to Such Absorption.—A piece of the natural earth is immersed in water, the quantity of water absorbed at different periods of time being measured. The penetration of the needle is also found. This is a comparative test, but also has immediate practical interest in regard to the change of the state of an earth when immersed by a dredging pond.

11.—Presence of Organic Matters.—This is merely a qualitative test to detect considerable quantities of organic matter.

12.—Rate of Settling of a Suspension of an Earth in Water.—This is designed to detect soluble salts or alkalies which may be present in an earth and which will affect the properties of the clay portion. For example, it is known that certain salts, such as gypsum, calcium bi-carbonate, iron salts, and others affect the properties of a clay by what is known as flocculation. These soluble matters cause the minute particles of clay to adhere to each other, to "flock together" to form an aggregate particle many times the size of the component particles. A clay thus flocculated tends to act more like sand or silt because the peculiar properties of clay depend on the fineness of the particles. On the other hand, substances such as sodium carbonate tend to deflocculate a clay. Whether a clay is flocculated or deflocculated is easily seen by its rate of settling in distilled water. This test would also have practical bearing with reference to the rate of settling in a fill.

Tests 13, 14, and 15, relative to percentages of voids in a natural earth, the ratio of the volume of water to the volume of voids in a natural earth, and the specific gravity of the particles, respectively, are comparative tests. It is also likely that Tests 13 and 14 will be found to be quite important as far as the working properties of an earth are concerned.

Proposed Method of Applying Data Secured

It is assumed that cost of work depends very largely on the daily output of the dredge and that all other cost factors are subordinate. On this assumption the following method of procedure is suggested: For example, select a fairly uniform bed of earth sufficient for the dredge, say for one week, or for whatever time is considered sufficiently long to determine accurately its daily capacity in the material. Sample the earth in several places so as to be repre-

sentative of material to be removed. The capacity of the dredge for this week would then be determined in the field, together with any special conditions encountered or properties of the earth which affect the ease or difficulty of working. In other words, the capacity of the dredge for the week and the ideas of the man in charge as to why the dredge had that particular capacity would be recorded. The samples would then be sent to the laboratory for testing.

This procedure would be followed as often as possible, the number of samples depending largely on conditions. That is, if the dredge had to dig in a strata of sand and clay during the week, the capacity would be influenced both by the sand and the clay, and there would be no object in taking samples of the mixture as the results would not be dependable, because it would not be known how long the dredge would be in the sand or the clay.

This procedure would give two sets of data, one from the field and the other from the laboratory. After a considerable number of samples had been thus tested and correlated with the field figures, it is believed it would be safe to begin predicting on samples taken from projected work. The procedure on projected work would be to sample the various kinds of earth and determine the properties of each. After these properties were determined, the results would be compared with those from samples on which work had already been done and on which the cost of digging had been ascertained.

It is probable that the properties will not match exactly, but curves can be established so that the values for new samples can be plotted and thus compared with old ones.

It will not be advisable to make predictions on new samples until a considerable number have been tested, as it is only by correlating the tests with known results of cost in the field that a foundation can be laid for foretelling the action of new samples.

Sampling of Earths

The samples should represent a body of earth of fair uniformity, should be taken from the earth in place, and should be put in tight containers so that no change will take place before the sample is tested. Samples for testing should not be taken to represent too large a body of earth because considerable variations may occur over large areas.

About 5 lb. of earth will be required to make the test outlined, and a 2-qt. tin can with a full width friction top, or some other tight-fitting top, will be satisfactory for carrying samples.

In more than three years since this plan of testing was begun, the writer's Company has had occasion to make estimates on earth work, particularly for hydraulic-fill dams, at widely separated points in North America, and has found the laboratory work to be of great help. Considerable success has also been had in analyzing materials to determine their suitability for fill purposes.

The most important factor determining the estimated cost of earth work by the hydraulic method is the probable output of the contractor's plant. In sluicing work the ability of the giants to excavate the material very often determines the output. In dredging, more often than not, the ability of the

dredge to excavate the material determines the output, rather than its pumping capacity. This is particularly true of hydraulic-fill dams as they generally have good foundations which means either rock, hard-pan, or clay, and it is hardly to be expected that such material as soft harbor mud, or loose, water-bearing sand, will be found in such locations. Wherever the output is dependent on the ability of the dredge to excavate the material, the study of earth samples as suggested is meant to apply.

A sufficient time has not yet elapsed to give a complementary set of records in the field and in the laboratory which will enable the Company to depend largely on the laboratory reports. It is too much to expect that laboratory reports may take the place of practical knowledge and experience, particularly as so many other factors have a bearing on the cost of work. It is expected, however, that in a few years the laboratory work will have been perfected, records will have been accumulated as to performance, and knowledge increased so that the laboratory will be a valuable adjunct to the earth-moving business.

If work is limited to a comparatively small field, particularly on tide-water, there may be no justification for a study such as suggested. However, the average dredging job is no longer a simple problem of excavating harbor and mud and pumping it a short distance, wasting the material, or, at best, making a simple fill with it. Broader knowledge is required by those responsible for hydraulic operations, and this, in turn, requires more dependence on laboratory research work.

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PROBLEMS IN CONCRETE DAM CONSTRUCTION ON THE PACIFIC COAST*

By Arthur S. Bent,† Affiliate, Am. Soc. C. E.

The first thought that arises out of a long experience in the building of dams is that no two of them are exactly alike and no one plan of operation is the optimum for more than a single location. This is true in spite of the obvious fact that different contractors might have different plans for the same work. Thus, it is evident that the success of an operation is measured by the correctness of what might be called its diagnosis. In the case of one notable California dam, the plan of an unsuccessful bidder for handling the work was so radically different from the one actually used, as to indicate definitely that they were not equally good and could not possibly produce anything like the same results in costs or speed.

It follows, therefore, that the first step in estimating, and by far the most important, is a painstaking, thorough study of the site and all its conditions and this is where the contractor's skill and vision are tested. If he is inexperienced in the building of dams he will probably learn much on his first job, but even with long experience he will be impressed with the truth of the opening statement that no two projects are alike. While remembering all the things that did not work well on previous work and all the unexpected conditions that arose, he must not be bound by precedent. He must have the ability to discard every type of plant and method of attack he ever used if they do not seem to fit, and to devise those that will. Standing on the untouched site with the plans before him, he must have the vision to follow through step by step the processes ahead and see clearly and in detail the methods, organization, and equipment required for each one.

After a lifetime spent in construction work of various kinds the writer believes this is more difficult to do with a Western dam than with any other

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structure. Compared with a Class "A" building, for instance, a dam seems very simple—just a block of concrete between two walls; but it is a significant fact that many fortunes have been sunk on single dams, while on others very low bidders have made good profits.

This paper is an attempt to suggest, from a long experience, the fundamental elements that must be carefully and intelligently considered in advance to make a success of a concrete dam, and to outline some of the methods of meeting specific problems.

Estimates

In the preliminary estimate the first thing to consider is the owners with which one has to deal. Are they responsible and fair-minded? are their financial plans sound? are they likely to use up in other developments the funds that should go into the dam before it is completed? What are the character, reputation, and experience of the engineers? An unfair, hard, or inexperienced engineer can make any job a misfortune to the contractor and take all the joy out of it without adding to the safety or usefulness of the structure. If a contractor has reason to believe that he will be in the hands of such a man, he should not bid—it does not pay.

The site should be visited many times and a long and thorough study made of it. The writer's method is to have independent studies and plans of operation made by different members of his organization, these being later reconciled in the office, so that the plan finally adopted represents the consensus of its best judgment.

These site studies must include accurate knowledge of the topography, water run-off, high-water period, weather history, water supply for entire year, length of working season, and possible methods of river diversion. Thought must be given to the effect on the plant and work of the inevitable floods and the work so scheduled as to reduce this misfortune to a minimum. Here, again, the co-operation or otherwise of the engineer is important. On one project the engineer needlessly delayed the placing of the concrete in the foundation with the result that an unexpected flood filled the excavation and cost the contractor a loss of $20 000. The remoteness and inaccessibility of most dams is an important matter. None of the ordinary facilities for securing skilled labor, supplies, repairs, and spare parts is at hand. The job must be constantly supplied with all these facilities and must be completely self-contained at all times.

Excavation

The estimate must take into careful consideration the character and proportions of soil and rock and their disposal after excavation. This seems too obvious to be worthy of mention, but it is probably on this item that errors of judgment are most common. Certainly there is greater difference in methods and plant for handling the excavating than for the concreting. On one of the writer's jobs it was believed that bed-rock was nowhere more than 14 ft. below the stream bed, but in one section it was found at a depth of 52 ft.

through large boulders and much water. This required a complete and costly re-adjustment of the entire excavating plant.

As a rule the estimated cost of excavating and wasting proves inadequate. The excavating constantly suffers all the disadvantages of the work. For instance, it is the reservoir for the poorest of the labor. Untried men are put in the excavating crew. If they prove extra good they are transferred to something else. Again, shooting of rock cannot be done in the most advantageous way because of the two dangers of injuring the plant and shaking the foundation. Also, other work is usually going on simultaneously and has the precedence, from which it happens that excavating frequently must be done at night, or in very restricted areas, both of which increase the cost.

Cleaning the bed-rock (which should be computed by the square foot) is slow and costly and although computed only once, frequently must be done several times. Here, again, the engineer is an uncertain element. It must be done to his satisfaction and this may mean removing loose stuff and hoseing off, or it may mean much hand-picking and scrubbing with wire brushes, and jetting with compressed air and water.

Wasting should be definitely planned and carefully computed. It is a frequent cause of loss. The quantity may far exceed that anticipated and thus bring in an entirely new problem of disposal, which possibility should be clearly provided for in advance.

Altogether the excavation on dams generally costs much more than other excavation jobs that look closely similar, and has proved to be abnormally difficult to estimate with accuracy.

Plant and Method

On these depend in the main the success of the work and final decision in regard to them is determined by many considerations. The time limit and transportation facilities affect the selection of plant almost as importantly as the type of dam and the topography. If there is no railroad, and the job is large and the time short, it will pay to build one. A railroad also should be provided usually if work is to be carried on through the winter. On a recent job, during which 400 000 cu. yd. of concrete were placed in 16 months, no trucks were used either at the dam or at the gravel pit which was 20 miles away. Where aggregates can be had near the work it sometimes proves economical to haul and store cement and equipment during the dry season. This would apply particularly to small dams in remote locations. The distances and quantities to be hauled should be accurately ascertained. On one of the writer's contracts a difference of $0.01 per ton on aggregates alone meant a saving of $10 000.

On high dams a hillside plant is desirable. This set-up takes full advantage of gravity, and reduces the flood hazard and the danger from falling rocks. However, when roads cannot reasonably be built to the top and there is room for both concrete and excavation plants in the stream bed, a tower is better. In all cases the concrete delivery point should be well above the dam crest and chutes should be amply large.

Low long dams frequently are best handled by trestle plants with the mixers at one end.

In general, although it is easy to "over-plant" a small job, it is highly important that the equipment should be fully adequate, always bearing in mind that rate of progress cannot be determined by dividing daily capacity into the total time allowed by the contract. The plant will produce its maximum output only in the middle of the construction. This further suggests that costs are increased by the fact that the same plant must be maintained throughout, while at the start and finish the output is small because of limited working space.

On a fair-sized job it pays well to make a solid and thorough set-up, using plenty of concrete for foundations of derricks, mixers, compressors, etc. The extra cost is more than compensated by regularity and certainty of operation.

The plant which is to produce the aggregate should be of ample capacity. Delays for materials are disastrous not only to costs but to morale, and in spite of seemingly careful preparation this very often happens. The writer's practice is to provide for an output fully 50% beyond the needs and on rush jobs to double the estimated demand, yet he has never overdone it.

If the rock is to be quarried near the work the usual estimating cost of $1.50 per yd. should be about doubled, and the manufacture of sand should be avoided at almost any cost of haul.

The supply of compressed air, power, and light should be ample and reliable and the last two should be electric wherever possible because of greater flexibility. A complete machine shop and a saw-mill are essentials.

Finally, the equipment should be not only adequate, but first-class throughout, and the supply of rails, pumps, tanks, pipe, cables, nails, clips, fittings, rope, wire, and small tools should always be kept well ahead of the needs.

Camp

Too much emphasis cannot be laid on the importance of this unit because the character of labor attracted to the work and its contentment and efficiency are largely the result of the living environment of the men. Quarters must be adequate, well lighted, heated, and ventilated. Sanitary regulations must be perfect, drinking water pure and palatable, the dining-room fly-proof, and the food first-class in quality and variety. There should be an ice plant, refrigerator, bakery, store for families, plenty of hot and cold showers, hospital with doctor and nurse in constant attendance, police and fire protection, post-office, telephone, schools, amusement hall, and recreation features. Different quarters should be provided for the different shifts as well as separate meal hours. As many as eighteen different meal hours may be required on rush jobs.

On one of the writer's projects the boarding-house plant cost $15 000. Incidentally, it was completely destroyed by fire early in the job. Notwithstanding the fact that the camp was on a side-line railway, with no road for automobiles, on the third day after the fire it was again in complete running order. Only two meals were served in the open and the construction work was never halted.

It is of prime importance that no reasonable detail be overlooked that will add to the comfort and well-being of the men.

A contented little city on his job is a matter of pride and satisfaction to a right-minded contractor. He should regard it not merely as a matter of enlightened self-interest, but as a fortunate opportunity to make life as good as possible for the group of men who, for a little while, come closely within his range of influence. He may never have that privilege again.

Organization

The superintendent must be selected with the utmost care and with particular reference to the work in hand. Splendid building superintendents can fail utterly on a dam. A rare combination of resourcefulness, speed, economy, executive ability, tact, good judgment, energy, and poise is required. He should also have technical training sufficient for the clear understanding of complicated plans, a first-hand intimate knowledge of all his equipment and machinery, and actual experience in rigging. He should of course neither drink nor gamble on the job, and his loyalty to his employer should outrun every personal consideration. An absolute desideratum, however, is the ability to plan every detail of the operation long in advance, without oversight or errors. He must never be caught unprepared by any detail of the work or by any emergency. His task is very similar to that of a military commander and his blunders may be as serious. Under the superintendent are carefully chosen heads of departments as follows: (1) Accounting and purchase; (2) commissary; (3) transportation; (4) excavation; (5) aggregates; (6) carpentering; (7) rigging; (8) mechanics; (9) electricity; (10) steel; (11) concrete; and (12) engineering.

Transportation

This has already been touched upon, but may be elaborated briefly as it is of great importance. A wrong guess on this feature at the outset may paralyze the work later. Provision must be made for handling materials rapidly and certainly and single pieces of equipment weighing as much as 75 tons. Motor trucks may be relied upon, but must be furnished with a first-class roadbed constantly maintained or they cannot give adequate and uninterrupted service. Almost any amount of money is well spent in securing reliable transportation.

Conditions must determine the best plan and these are never twice alike. On one of the writer's projects, everything had to be transported through a 5 by 7-ft. tunnel, 4 miles long. This compelled the cutting and re-welding of large mixer-drums and similar heavy units. Where the yardage is comparatively small and the country extremely rugged tramways can be used to advantage. On major projects such as the Exchequer and Hetch Hetchy Dams, standard gauge railways are indicated, and they should be operated as systematically as main-line roads.

Diversion of Water

In the diversion of the water the constructor of dams faces one of his most serious problems. A mistake of judgment may cost him not only great dam-

age to his plant and work, but the loss of a whole season's work. On one project it was stipulated in the contract that such a delay would damage the owner $500 000. Nobody prayed for rain on that job.

The problems involved in handling the water are many, for the bottom may be solid rock, or it may be quite pervious and the run-off may vary during the life of the job from a few to tens of thousands second-feet. Diversion is accomplished by tunnel, flume, wing-dams, or an open section through the dam, controlled by gates. Pumping must be expected and its cost is nearly always under-estimated. If the rock is fissured, pumping may become a heavy expense. When coffer-dams are used they should be placed far enough from the site to be safe from injury by blasting.

Demolition

Demolition is a trying part of the work and unless exceptional vigilance is exercised and the morale of the crew carefully maintained it will show loss and waste. The writer's system is to sell at once as much of the plant as possible, and on major projects a yard with railroad facilities is rented for the temporary storing and handling of equipment. All equipment, whether to be sold or retained, should be put in first-class order and painted without delay, and all loose parts should be marked to prevent confusion and loss.

The energetic handling of this feature may determine the entire outcome, as it is not uncommon to find on a heavily "planted" job that its profits must come out of salvage. This may be regarded as one of the results of unintelligent competition.

Dam building is at once one of the most interesting and precarious types of construction. In spite of experience and care there is constant hazard from conditions beyond control. On the great majority of the writer's jobs floods have repeatedly submerged the work. Temperatures range from snow to 115° Fahr. on single projects. In one case a mountain side above the dam slid down dumping 25 000 cu. yd. of rock on the concrete. In two days the contractor's steam shovel and narrow-gauge railway working double shift cleared it away, and this equipment had to be taken 200 ft. up the vertical face of the mountain. These things suggest the necessity for a liberal contingency item in the estimate for unforeseen and uncontrollable emergencies.

The interest in dam building lies in the fact that conditions are never the same and the ever-changing vicissitudes of the work call forth almost daily the utmost resources of the constructor. More than in any other type of structure, the final outcome depends on his foresight, energy, and courage. A big dam is always a splendid adventure in construction.

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CONSTRUCTION METHODS ON THE MOFFAT TUNNEL

Discussion*

By Lazarus White, M. Am. Soc. C. E.

Lazarus White,† M. Am. Soc. C. E.—A project such as the Moffat Tunnel has three aspects: (1) Economic; (2) engineering; and (3) contract relationship. It is difficult to understand its economic aspects, particularly as it is being built previous to an agreement with the only railroad that can operate it. Although this is quite the usual practice, it often leads to a poor bargain on the part of the community that originally finances such a project. Of course, the building of this tunnel is a fine thing for the railroad, which secures a great bulk of valuable freight during its construction and, later, will be in a position to drive a good bargain. Perhaps the community takes its profit in the opening of a new district west of the Rocky Mountains.

For building the Moffat Tunnel a new method of contracting was devised; the contractor is compensated at a small fixed fee set by himself, the main incentive of profit coming from completion ahead of time, thereby securing a bonus. Because of the difficulty in driving the tunnel, it appears that the bonus will be hard to realize and that the contractors, despite their very excellent work, experience, and abilities, will not receive much compensation, unless it is through sources not apparent in the original contract. This form of contract is, in fact, a method of hiring superintendence. In the long run, it would practically eliminate contractors, as ordinarily understood, and would substitute for them professional men in a sort of dual capacity of engineers and superintendents, operating on a salary with an incentive of a bonus, but lacking the element of profit, which is the principal attraction for the type of man who usually makes a successful contractor.

The engineering problems are those which have repeatedly arisen in connection with the driving of other long mountain tunnels of Alpine character,

* Discussion of the paper by R. H. Keays, M. Am. Soc. C. E., continued from May, 1927, *Proceedings*.

† Pres., Spencer, White & Prentis, Inc., New York, N. Y.

such as that at Rogers Pass. The difference between these two tunnels, however, is due to the presence of, or freedom from, heavy ground. The methods of overcoming heavy pressures are interesting and ingenious. Apparently, the construction was hampered by the difficulty of securing steel, cement, and concrete materials so that resource was had largely to heavy timbering. In the writer's opinion such problems could better be met by steel and concrete.

In such cases timber is not a reliable material and cannot be trusted to hold heavy pressures permanently. The failures noted in this paper are similar to those historic examples in Drinker's "Tunneling". Nowadays, because steel is readily available and methods of placing concrete by compressed air have been developed to a high degree, bad ground can be more readily supported by steel and permanent concrete lining in the writer's opinion.

This paper is valuable inasmuch as Mr. Keays very frankly states his problems and expresses doubts that the solutions reached were the best. Such an attitude leads to progress. In describing a completed construction, there is a tendency to give the impression that the methods used were the best, thus leading others to copy examples that are questionable.

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AUTOMOBILE HAZARD IN CITIES AND ITS REDUCTION

Discussion*

BY MESSRS. W. W. CROSBY, THEODORE T. MCCROSKY, AND R. S. KIRBY.

W. W. CROSBY,† M. AM. SOC. C. E. (by letter).‡—The proper location of highways, the writer has long contended, is fundamental to reducing or relieving many highway difficulties, including those of highway safety. Location shares with zoning in a recognition and understanding of the kinds of traffic to be provided for; it comprehends, as does planning, the automatic segregation of traffic; the avoidance of congestion when and where necessary; the question of speeds; and the widths of highways, as well as alignment of the roadway.

As to "hazards" and "safety", at this time when such a highly scientific consideration of a common public matter is being carried on, a note of warning should be sounded. People should remain unemotional in considering this matter, strongly as they may be impelled for a while to personalize "safety", in order to bring home to the Nation, through the individual, the need for action to attain a lasting public benefit.

"Safety first" does not seem to convey the meaning elsewhere that it has in the United States. In America, highway safety began with the "watch-your-step" injunction of the New York subways and surface railways. That this had as a basis the safeguarding of the financial interests of these corporations against liability for personal damages can hardly be denied. There followed the somewhat broader "safety first" with its personal and individual appeal.

The Parisian practice of arresting the pedestrian run down by a vehicle in the streets of that city, and charging the unfortunate with "disturbing traffic,"§ may sufficiently exemplify the basis on which European ideas of

* Discussion of the paper by William J. Cox, Jun. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Cons. Engr., Baltimore, Md.

‡ Received by the Secretary, May 9, 1927.

§ Highway Code, Article 55.

safety on the public roads seem to rest and from which foundation the present attitude has been developed in the European capitals, where most highway problems are older and, in some ways at least, better understood.

A study of the reports of the Fifth International Road Congress (Milan, September, 1926) impresses one with the recognition, apparent in the minds of the higher road authorities, of the public interest in the secure (safe) and steady (economic) flow or circulation of all traffic over the highways. The thought and effort seem to be toward "the efficient utilization of the highways with the greatest possible integrity of the most rapid circulation practicable"; first, by means of engineering expedients, and, second, when necessary, by supplementary general regulation. As long as it is consistent with the ends in view, the public is left unhampered in the choice of its means of highway use. Interruption of the personal choice of transportation, or other use of the highway by the individual, is imposed only where reliance on the individual responsibility, for his own and the general welfare, cannot longer be satisfactory to the community.

Mr. Cox has given a scientific treatment of what is so often regarded as a psychological or sociological problem. He brings out with astonishing clarity the functions of the engineer and proves his theories with intelligible mathematics. The elimination of some of the indefinite factors in these problems and the resolution of others into mathematical equivalents strike the writer as both logical and ingenious—well worth imitation and development.

THEODORE T. MCCROSKY,* JUN. AM. SOC. C. E. (by letter).†—In reducing the complex problem of automobile accident hazard to mathematical terms, the author has found a formula which gives a gratifying agreement with the "actual hazard", as determined by insurance companies for rate-making purposes.

The author deduces that, from a public liability standpoint, the hazard is proportional to $\frac{P}{M}$, for any given city, and hence is independent of the number of automobiles in the city. Some cities have a large concentrated population, but very few automobiles. The value of the hazard, H, would be high, as given by the formula, $H \alpha \frac{P}{M}$. There are many such cities in Europe, in which, automobiles being scarce, the accident hazard is necessarily low.

Mr. Cox has proved that, for 1922, street width, good motor vehicle laws, and efficient handling of traffic are quantities which appear as mutually compensating in the majority of American cities of more than 200 000 population; and, consequently, need not be taken into consideration as variables in the problem. In the case of the other cities, he has taken up these three auxiliary factors as accounting for discrepancies between calculated and actual hazards. In the practical application of his formula, it would be hard to predict when these variables would compensate.

* Instr., Sheffield Scientific School, Yale Univ., New Haven, Conn.
† Received by the Secretary, July 5, 1927.

The writer believes that these three variables, as well as the registered number of automobiles, should appear in the formula, because all four of these considerations have changed greatly since 1922, and are becoming increasingly important.

Streets are being widened in the congested parts of cities where sidewalk widths permit. There are rigid motor vehicle laws in many more States than the four listed in 1922. The scientific regulation of traffic has increased enormously, but it is far more efficient in some cities than in others. Since 1922, the automobile registration in the United States has increased more than 80 per cent.

The writer fears that the author, in his praiseworthy attempt to reduce the problem to simple terms, has somewhat overshot his mark by omitting from his analysis these unknowns which, unfortunately, should no longer be neglected.

R. S. KIRBY,* M. AM. SOC. C. E. (by letter).†—The author has made a keen analysis of a pressing and intricate problem, from an actuarial standpoint, using a type of mathematical reasoning not so familiar to the average engineer, and the result comes out as a formula. Engineers delight in formulas, but are chary about using them unless they are reasonably familiar both with their antecedents and with their consequences.

Concerning the author's basic formula, $H = \frac{P}{M}$, a number of facts should be kept clearly in mind:

First.—The whole discussion has to do with fairly large centers of population, where less than one-half the automobile accidents occur. In less thickly settled localities this problem assumes quite a different aspect; there are fewer street intersections, and a much smaller proportion of pedestrians to cars.

Second.—The author centers his discussion about automobile accidents in which pedestrians and automobilists are injured or killed. An accident in which both cars were hopelessly wrecked, if providentially no one was injured, would not figure in his analysis at all. Neither, apparently, would an accident in which the driver was killed, or any of the occupants of his car. This is, of course, confining the argument within narrow limits and excluding a large percentage of what are commonly thought of as auotomible accidents. For example, in 1926, in the entire State of Connecticut, out of every 100 accidents reported to the Motor Vehicle Commissioner, 65 involved damage to property only and 35 resulted in injury to a person, either in a car or on the street. Of all the accidents reported, only about 15% were collisions with pedestrians. Of the persons killed in Connecticut in 4 years, 55% were pedestrians. The percentage of pedestrians among the injured was about 45. Of course, in large cities, all these percentages are quite different.

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† Received by the Secretary, July 20, 1927.

Third.—Dealing with the problem as one of insurance, the author entirely neglects the large number (about 50%) of uninsured cars, dismissing them with the statement* that "there is no reason to believe that the hazard created by an insured car is materially different from that created by an uninsured car; * * *". Of course, it is yet too early to draw any certain conclusions from experience with compulsory insurance.

Fourth.—Concerning the natural deductions from the $\frac{P}{M}$ formula, it is, of course, not logical to assume that the expression holds true for all possible values of the variables. If P were zero, as in remote country districts, there might still be automobile accidents, from cars passing through. Accidents caused by out-of-town and out-of-State cars form nearly one-half the total each year in Connecticut towns. The writer understands that another implication, and a justifiable one, is that if P were doubled, the hazard per car would be doubled, but the total number of accidents would be quadrupled. If the miles of improved road could be doubled over night, the number of accidents to-morrow would not, necessarily, be altered at all. This is partly because, as the author states, "a majority of collisions between automobiles in the average city occur at a comparatively small number of intersections." What effect, then, could a sudden increase in mileage in outlying sections have on traffic hazards in crowded tenement districts?

From his own experience the writer can thoroughly appreciate the author's difficulties in securing, from different localities, accident statistics which are exactly comparable. For example, the author's discussion considers only the number of injury accidents. The number of persons injured may be considerably greater and is more usually recorded.

What is a motor vehicle accident? The various States are not uniform in their definitions. Connecticut, for example, requires the reporting of all accidents which result either in any personal injury or in property damage to an apparent extent of more than $10. Other States neglect property damage accidents entirely. This illustrates only one phase of the difficulties in securing comparable statistics from various cities. Uniformity in this matter is a long way off, and some of the statistics that the author needs in order to substantiate his reasoning are available only in a very few cities, if at all. Engineers are natural born statisticians and can help in keeping such facts before the public.

The author says emphatically that the variation in hazard between different cities is not caused by differences in automobile congestion or in the effectiveness of traffic control. However, in the case of Boston, Mass., may it not perhaps be that both these factors contribute, each in its own way, to reduce the local hazard?

The latter portion of the paper, in which certain methods of reducing accidents are most concisely outlined, should have been expanded. For example, the main motor-vehicle arteries of the future will not pass through the centers of any towns. This involves intelligent engineering, and is vastly

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 516.

more important than much of the minor puttering and patching which is being done. A decided reduction in accidents can be brought about by intensive local public campaigns designed to crystallize public sentiment against three practices. Of these, the first is speeding, or driving too fast for conditions. The second is unnecessary driving in congested areas, or using the public highways for trivial errands. The third is a lack of consideration for the other fellow, be he on wheels or on foot. While these three practices are not engineering matters, nevertheless, the engineer, as a private citizen, can do much in his quiet methodical way to better conditions.

All the author's suggestions concerning the use of streets as playgrounds, particularly in tenement districts, are reasonable. Autoists of the future must, perforce, give up many more of their "rights" for the common good.

Drivers are responsible for three-quarters of all automobile accidents (including collisions between cars), at least that is the conclusion of the Connecticut Motor Vehicle Department which has had long experience in determining primary causes. When drivers are trained to concentrate their attention on their driving, to have a decent regard for ordinary rules of the road, and to drive so carefully that their cars need never skid, two-thirds of the automobile accidents will have disappeared.

The State of Connecticut held pedestrians to have been at fault in 74% of the pedestrian collisions last year. The 1926 percentage in Massachusetts was estimated at 37 for non-fatal collisions and 25 for fatal, a wide divergence, due probably in large part to varying methods of analysis.

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PROBABILITY OF FLOOD FLOWS

Discussion*

BY MESSRS. R. W. DAVENPORT, R. D. GOODRICH, AND ALLEN HAZEN.

R. W. DAVENPORT,† M. AM. SOC. C. E. (by letter).‡—In studying some engineering problems, it is pertinent to inquire how to extend the application of statistical science only so far as the advantages are commensurate with the labor involved. In other words, how may one obtain the most light on a subject without introducing too many of the somewhat intricate statistical processes? This applies to the study of the frequency of occurrence of unusual flood flows. For example, given a typical discharge record covering 20 years, it is desired to estimate the maximum mean daily flow likely to occur at an average interval of once in 100 years. The desired type of flood flow series may be selected from: (1) The annual maxima; or (2) the peak discharges above some lower limit (such as the minimum annual peak), each one returning to the minimum annual peak; or (3) all discharges above a certain minimum discharge, etc. Somewhat laborious calculations based on statistical theory may be made to determine the appropriate curve to represent the selected series. On probability paper, if a skew probability curve, or on logarithmic paper, if of an appropriate type to be shown thereon, it will give a good representation of the series from the standpoint of frequency of occurrence. On the other hand, at an expenditure of considerably less labor, the series may be plotted directly on either of these ruled forms in accordance with the frequency of occurrence, and a mean curve may be located quite readily through the points by eye. Giving the general question specific application to this case the writer inquires whether the projection of the calculated curve to show the 100-year flood discharge is likely to be

* Discussion of the paper by F. G. Switzer, Assoc. M. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Hydr. Engr., Federal Power Comm., Washington, D. C.

‡ Received by the Secretary, June 29, 1927.

enough more trustworthy than the projection of the curve put in by eye, to justify the additional labor involved in obtaining it?

The selected 20-year flood flow series may be regarded as a sample short series selected at random from a long series. How nearly is the sample short series likely to represent the long series? Yule presents* a mathematical method for determining the probable error of a percentile where the sample series contains a large number of terms, but because of the small number of terms in the case assumed it is obviously inapplicable except as to the general conclusion "that the influences of the fluctuations of sampling on the several percentiles increases as we depart from the median." The sample short flood series is likely to represent the long series much more closely near the median flood than it is so far therefrom as the 100-year flood, especially when the latter is determined by extrapolation. The writer is not aware of any satisfactory theoretical method of determining the probable error of a 100-year flood estimated from, say, a 20-year record, but it is inferred that such error would be quite large whether estimated by using a calculated skew curve or a curve put in by eye.

E. W. Lane, M. Am. Soc. C. E., compares† flood probability curves determined from different parts of the 72-year record of Merrimack River. It is assumed that this is a reasonably typical record. The data presented show that the maximum floods for the four 18-year periods were 113, 111, 91, and 78%, respectively, of the flood, with an average interval of 18 years as determined from the entire record. Thus, in this case, the average discrepancy of the flood maxima of the 18-year periods in representing the 18-year flood of the 72-year record is 14 per cent. It is likely that curves showing the several series would have discrepancies of this same order in representing the 18-year flood. If the probable error of an 18-year record in indicating the flood with an average interval of 18 years is of the order of several per cent., what is likely to be the probable error of a 20-year record in indicating the flood with an average interval of 100 years? Is it not likely to be so great as to make the projection of the curve representing the series as made by eye essentially as accurate as the projection of the computed skew curve?

There is no question but that flood flow series follow skew probability curves in a general way, and consideration of the somewhat accidental nature of the factors producing floods makes it seem reasonable that they should. However, some of the factors may have a very limited range of operation, which condition could not be adequately taken into account in the computation of the skew curve, so that to this extent the computed curve would fail to represent the series. For example, assume two drainage areas alike in every respect, so far as flood-producing factors are concerned, except that the floods of one pass through a well-defined channel with banks so high that it never overflows, whereas the floods of the second pass through a channel like the other for ordinary floods, but for high stages overflowing a wide area so that the flood-plane storage becomes a material factor in holding down high floods. Below the stretches of channel considered, the frequency dis-

* "An Introduction to the Theory of Statistics," by G. Udny Yule, p. 337.

† *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1059.

tributions of floods of the two streams would be the same except for high floods, where, because of the introduction of flood-plane storage on one stream, they might be appreciably different. This difference would not be accurately reflected in the computed curves. Is it not probable that a projeection of a curve estimated by eye beyond the limits of available data could take such factors as that illustrated into account equally as satisfactorily as the projection of a computed skew curve?

The writer does not wish to be understood as giving too low an estimate to the value of the calculated curves. The calculations have the advantage of subjecting the data from different streams to uniform treatment and doubtless general principles of some importance can be developed thereby. If the subject of the study is the more frequent flood flows, the calculated curves might often give more satisfactory results than the curves located by eye. Both methods have a proper place in flood flow study.

In Table 1* the author presents an interesting comparison of the flow to be equalled or exceeded in certain time periods as estimated from: (*a*) Annual peaks for the year beginning October 1; (*b*) the daily record; and (*c*) flood flows above a certain limit. It is not clear to the writer how the author selected the peak flows of the storms in Item (*c*). In whatever way they were selected it would seem that the mode for the series would be at at the lower limit and the series would be essentially only a part of a complete series.

Probability paper is adapted to plotting data with the mode at or near the median and according to the writer's understanding the Foster method of computing skew curves to represent statistical data assumes that the data cover the complete series or the "universe" of the combination of factors producing the individual members of the series. Therefore, the writer wonders whether logarithmic paper, rather than probability paper, would not be better adapted for the graphical representation of the series in question.

The author shows the values for Items (*a*) and (*b*) as diverging somewhat as the time period increases. As a matter of fact the two series when plotted against the time interval would join at the maximum discharge for the period under consideration. This is a significant fact that might well be taken into account in the extension of curves based on short records. If the period to be represented were very long, the extensions would apparently be nearly parallel and converging rather than diverging. It seems probable that for the longer time periods the two extensions should give nearly the same values. Incidentally, this would seem to be a characteristic of the data which can be taken into consideration more satisfactorily by the graphical method than by the calculation of theoretical curves.

There would be some advantage if engineers would agree on a uniform method of selecting flood series to be used for showing the general flood characteristics of streams. The annual maxima and the part of the discharge duration table or curve representing the high discharges, when taken

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 564.

together, give a reasonably comprehensive index of the flood characteristics of streams. Furthermore, the annual maxima and the duration table afford satisfactory data for treatment with respect both to application of statistical principles and to deduction of hydrologic characteristics of streams. In a study of the floods of a particular stream or system of streams consideration of the data from various angles other than the two mentioned would very likely be desirable.

R. D. GOODRICH,* M. AM. SOC. C. E. (by letter).†—The writer has been applying frequency curves to the study of flood flows for some years, and, therefore, is very much interested in Mr. Switzer's description. Although some statisticians seem to believe that these curves will not help to predict the magnitude and probability of flood run-off when records for no more than twenty or thirty years are available, others believe that they may be most valuable aids.

In the flood studies of Chihli Province, China, the annual maximum flows, peak flows (above an assigned minimum), and daily discharges were plotted as shown in this paper. The results are not likely to be the same on account of the difference in the methods of selecting the "samples" and still more on account of the difference in the number of items in the several samples. For frequencies well beyond the actual range of the observations, however, the results in Table 1‡ may be quite consistent when considered in the light of the probable errors of the resulting discharges.§

The author calls attention to the effect of "wet" and "dry" years, but does not seem to allow for this in using curves drawn with all peaks above a certain assigned limit. There is some difficulty in interpreting such curves. During "dry" years the "time interval" may be two or three times as long as the interval between flood peaks in a "wet" year, so that the proper time interval for use with the higher values of floods may be quite different from the average value assumed by the author. When annual maximum flood peaks are used for defining a frequency curve, the time may be properly expressed in years, but it is somewhat questionable whether this can be done when all flood peaks are used for plotting the curve, unless there are approximately the same number each year. A safer expression would seem to be in terms of the percentage of floods greater or less than any specified value instead of the percentage of time that this value may be exceeded or not exceeded. If maximum floods are possible in any month of the year, the maximum daily flow for each month during the years of record may sometimes be used with satisfactory results.

The data used for the results given in Table 2‡ cannot properly be defined by a single frequency curve. A sample of 294 items is fairly large and, if

* Engr. in Chg. of Works, Chihli River Comm., Tientsin, China.

† Received by the Secretary, June 30, 1927.

‡ *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 564.

§ A formula for the probable error in the computed value of the variable for any given frequency, is given in the closing discussion on the writer's paper on "Straight Line Plotting of Skew Frequency Curves," which will appear in *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 100, to be issued in the early fall.

taken from homogeneous material, the omission of two of the largest should not cause such a decided change in the shape of the curve. It would seem more probable that there are two or more sets of causes producing floods on the river in question, and that if the data were used to draw two curves, the results might be consistent.*

Unfortunately, none of the data is given in complete tabular form so that it is impossible for one to make an independent, comparative analysis by any other method. With only about forty points to represent several hundred items, or even about one hundred items, the scale of Figs. 1,† 2,† and 3,‡ is too small for satisfactory work, although such a comparison would have been of particular interest.

Perhaps the most interesting and suggestive point in the paper is the hint "that floods of equal probability are proportional to some function of the drainage area". If further study proves this to be true, it will add very much to the usefulness of Mr. Jarvis' method and formula for the study of flood flows.§ The author is to be commended for bringing this use of frequency curves more forcibly to the attention of the profession.

ALLEN HAZEN,‖ M. AM. SOC. C. E. (by letter).¶—The author finds that selecting different ends of the water-year will give different results. This is one of the things that happens with short-term records. The longer the record, the smaller this difference would be. Using the calendar year for the water-year in all cases has advantages, and experimenting with variations is hardly warranted. It must always be remembered that short-term records only give rough approximations and precision cannot be obtained from them by any procedure however perfect.

The values that represent either maximum flood rates, or maximum average 24-hour floods, selected one for each year, form a series that can properly be subjected to the analysis used by the author; but when all floods above a certain arbitrary limit are chosen, regardless of the years of their occurrences, it may be questioned whether the numbers thus found, constituted such a series as can properly be examined in that way.

With the 28-year record referred to, these values, to the numbers of 40 or 50, or any other number of terms, really represent that number of the highest terms, in a series of 10 227 terms, equal to the whole number of days in the record period. The entire series of 10 227 terms, if collected and arranged, forms what is commonly called a duration curve.

This is a useful curve and a study of it may lead to valuable results. There is no reason why the highest terms should not be separated for special study, but plotting them across the entire range from 0 to 100% of probability

* One or two examples of this kind are given in the closing discussion of the writer's paper referred to previously.

† *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 566.

‡ *Loc. cit.*, p. 567.

§ "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

‖ Cons. Engr. (Hazen & Whipple), New York, N. Y.

¶ Received by the Secretary, July 5, 1927.

paper is hardly the best method of procedure. The author's diagrams are sufficient proof of this. If these results are to be plotted on probability paper they ought to be moved over, extending the scale of the paper to accommodate extreme values, and plotting them in their proper positions as the end of a very long series. The decimal positions of the first few terms would then be, 0.000049, 0.000146, 0.000244, etc. The method must be entirely graphic and a line may be drawn to represent the points on almost any sort of a diagram.

Weston E. Fuller, M. Am. Soc. C. E., in 1914, used semi-logarithmic paper,* and as a practical matter it is almost as good for use in this particular way as probability paper, but in other cases the probability paper is preferable.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

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THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

BY V. R. COVELL AND T. J. WILKERSON, MEMBERS, AM. SOC. C. E., AND A. D. NUTTER, ESQ.

V. R. COVELL† AND T. J. WILKERSON,‡ MEMBERS, AM. SOC. C. E., AND A. D. NUTTER,§ ESQ. (by letter).‖—From the standpoint of economy, rigidity, and general structural efficiency, the Florianopolis Suspension Bridge is a splendid type, having advantages that cannot be questioned. The authors' claim for its esthetic value is not so easily established. They state¶ very truthfully, that "esthetic values in structural designs are generally unsatisfying on account of the existing differences of individual taste." Very rarely, in fact, have those who are considered competent to judge, been in agreement.

The design of L. S. Moisseiff, M. Am. Soc. C. E., for a bridge across the Kill van Kull** gives a more satisfactory outline, in that the side spans are supported by the back-stays and the same type of stiffening truss carried from end to end of anchorages. This gives a desirable continuity to the structure, which is lacking in the Florianopolis Bridge, with its straight back-stays and separate deck viaduct construction for the side spans.

Both bridges are more pleasing in an outline drawing than in the finished structure because the actual widths of members influence the appearance so greatly. As viewed in the field, the wide built members stand out boldly (particularly the top chord of the stiffening truss), while the eye-bar chain

* Discussion of the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Chf. Engr., Bureau of Bridges, Dept. of Public Works, Allegheny County, Pittsburgh, Pa.

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‖ Received by the Secretary, July 21, 1927.

¶ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 720.

** *Engineering News-Record*, November 27, 1924, p. 883.

and wire rope hangers seem to be much more faint and tend to disappear entirely when viewed from a distance. This results in an outline profile which every one except the members of the Engineering Profession finds rather difficult to understand and reconcile with such a long span. The appearance of this type of suspension bridge would be greatly improved with a slight sacrifice in economy, by using a light false member to maintain the full width of the top chord of the stiffening truss over the part where it is combined with the eye-bar chain.

The weight of steel in the Florianopolis Bridge, given as approximately 4 400 tons, is surprisingly low for a structure with a main span of more than 1 100 ft. and a total length of about 2 700 ft. This is an average of less than 3 300 lb. per ft. of bridge and approaches. The following factors account mainly for the lightness of the structure: (1) The use of a timber floor with the consequent saving of dead load; (2) the use of high-tension, heat-treated carbon steel eye-bars for the chain; and, (3) the economy of metal in the type of span used.

The first factor makes impossible a direct comparison with highway bridges, as built in the larger centers of population in the United States. The practice of using timber floors is fast becoming obsolete, especially on bridges in highly congested centers; but where the traffic is rather light, causing relatively little mechanical wear, and where a minimum cost is the outstanding requirement governing the design, a timber floor is undoubtedly the most satisfactory type to adopt.

During the past few years there has been a great increase in the loads passing over highways and these must, necessarily, pass over some of the bridges. While floor systems are being designed for 18-ton trucks, very frequently electrical equipment and road machines, weighing from 26 to 35 tons, must be accommodated. The legal weight for trucks using highways in Pennsylvania is 13 tons. It would be interesting to know whether the 6-ton truck used for the highway loading of the Florianopolis Bridge is the maximum load anticipated. The use of 4-in. plank on a stringer spacing varying from 3 ft. 9 in. to 4 ft. 6 in., apparently contemplates no increase and no exceptional loads.

A question arises as to the advisability of using high-tension, heat-treated carbon steel for bridge members. On the Florianopolis Bridge it was specified that this material should meet the following requirements:

Ultimate strength	105 000	lb. per sq. in.
Elastic limit	75 000	"
Minimum elongation in 18 ft	5	per cent.

Both the elastic limit and ultimate strength are easily controlled by the carbon content and heat treatment, but with a sacrifice of ductility; and a favorable combination of the first two characteristics gives no positive information as to what may be expected in the third. For ordinary carbon steel, annealed eye-bars, the minimum elongation in 18 ft. is generally specified as 12% and for heat-treated eye-bars having an elastic limit of 50 000 and an ultimate strength of 80 000, as 8 per cent. While most of the test bars give

an elongation of more than 6%, one gave the very low value of 3.8%, and it is quite possible that a number of bars used in the chain, if tested, might run below 5 per cent.

Nickel, manganese, and silicon alloy steels combine the desirable qualities of high tensile strength and ductility and should undoubtedly be given careful consideration in conjunction with high-strength, heat-treated bars for any particular condition, where high ductility is necessary.

The designers of the Florianopolis Bridge have used the proper material for the eye-bar chain, the only question being, what is a satisfactory minimum value for ductility? It seems as if 5% were sufficient, but that the higher values should be used for truss members.

Following a suggestion of the American Bridge Company, the use of heat-treated eye-bars was adopted by Allegheny County, Pennsylvania, in the design of all large county bridges constructed since 1924, or now under construction, with unit stresses somewhat lower than those used in the bridge under discussion, all such eye-bars being tested by the Brinell hardness test to insure uniformity of treatment. By taking lower unit stresses, it is possible to use bars without thickened heads, and, in many cases, to use ordinary carbon steel for pins. These heat-treated eye-bars have been used in three bridges of the suspension type, in three of the cantilever type, and in two of the simple truss type.

The hinged tower for the suspension bridge is a step in the right direction, and this design was adopted in the 6th, 7th, and 9th Street Suspension Bridges over the Allegheny River at Pittsburgh, Pa.

The elongated pin-hole in the eye-bar heads leaves an opening in front of the pin. Was it considered necessary to fill this space to exclude water and prevent corrosion at these points?

The authors have met all the limiting conditions imposed on them in the development of the design of this structure, and it is an outstanding example of a large project consummated in a satisfactory manner with a limited amount of material and funds.

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SOME PHASES OF IRRIGATION FINANCING

Discussion*

By Messrs. F. H. Newell and J. C. Stevens.

F. H. Newell,† M. Am. Soc. C. E. (by letter).‡—This paper, issued to the public on practically the twenty-fifth anniversary of the passage of the Federal or Newlands Reclamation Act, marks an epoch in Federal relations to internal development. During the quarter of a century that has elapsed there has been a notable change, almost a reversal of attitude, toward reclamation ideals. A broader interest is being shown on the subject of Federal financing of reclamation enterprises. Inquiries are being made as to the propriety of spending the taxpayers' money for such purposes.

The paper throws a ray of light upon what has been an exceedingly sensitive, not to say a sore, spot; one which politicians and promoters have tried to conceal or from which they have endeavored to distract public attention. In one sense it has been forbidden ground. Engineers of ability have not been continued in Government service because they have been too outspoken or, in the words of the politician, have "not been practical"; meaning that the so-called practical needs of a party group has not outweighed their loyalty to the best interests of the whole country. Their mistakes were in not quietly acquiesing in unwarrantable expenditures. They were prone to paraphrase a classic, saying, "Reclamation, how many economic crimes are committed in thy name!"

The crucial point is summed up in the statement, with certain qualifications, that State and National subsidy to irrigation is necessary; that it is justifiable and desirable; and that large reclamation works "will not be undertaken by unaided private enterprises." To put it another way, large-

* Discussion on the paper by D. C. Henny, M. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Pres., The Research Service, Inc., Washington, D. C.

‡ Received by the Secretary, July 21, 1927.

scale irrigation or reclamation of waste lands does not pay under existing conditions. In a similar sense it has rarely paid. Comparing the actual cost of reclamation with the productive value of the land reclaimed, making fair allowance for interest, depreciation, and overhead, the facts stand out that these great enterprises, with few exceptions, always have been and always will be financial failures. This point has been avoided or obscured by interested parties and a review of 25 years of experiment and research is just bringing it out. It is an anathema to the "practical" politician seeking larger appropriations of Federal funds. It is the rock on which otherwise brilliant engineering reputations have been wrecked and where the "powers that be" have erected a warning to "keep off; discussion of real costs may be followed by dismissal. The door is always open for your departure."

Facts have an unpleasant way of asserting themselves. The author has made a plain analysis of the situation and has brought forward again the statements presented in 1924 by Mr. R. P. Teele, notably those in *Bulletin No. 1257*. Until the present time, this *Bulletin* has not been given the appreciation it deserves.

Twelve years ago the writer tentatively presented a discussion of this matter under the title of "Economic Advisability of Irrigation",* but the time was not ripe. Striking examples were not available without violating official courtesies. It has been necessary for the "disease to run its course" so as to establish beyond dispute, facts now available to all who care to go into the broad subject of reclamation finances.

In order that what follows may be clearly understood, emphasis is placed on the fact that there is no apparent reason for disagreeing with Mr. Henny's qualified statement† that "State and National subsidy to irrigation is justifiable and desirable".

The one fact in this connection that should be made clear, on which the spotlight should be continually kept, is that there should be no secrecy nor camouflage with respect to the amount and character of this subsidy. It is of interest to the taxpayers. The public in general should not be kept in ignorance nor misled as to the extent to which the taxes are increased, directly or indirectly. The subsidy is not wrong in itself; it may be a good thing; but it should not be concealed.

No culpability can be attached to the payment of necessary subsidies to irrigation or to other forms of reclamation, provided the public in general knows how the money is being used. No excuse should be tolerated for the activities of those interested in obscuring the facts; more particularly as in so doing, attacks are made on the ability of the engineers who have planned and executed the works of reclamation.

A condition has existed that has gradually aroused the indignation of engineers and lovers of fair play. This has come about from the fact that, in order to justify or secure the necessary subsidy of reclamation works and

* *Transactions*, Inter. Eng. Cong., San Francisco, Calif., 1915. Waterways and Irrigation, Paper No. 35, pp. 371–397.

† *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 912.

to bring about a partial repudiation of the monies advanced for them, the easiest course of procedure has been to attack the engineers under whose immediate direction the money was expended. The story (a reflection on local political ethics) is simple, the details are familiar to those who have followed the history of Federal reclamation development.

The first chapter is that of eager importunity on the part of local land owners and their representatives for the expenditure of Federal funds. This is accompanied by a statement that no matter what reclamation costs the lands will be worth it and the owners will cheerfully repay it.

The second chapter is of popular enthusiasm, praise of the engineers and their operation, accompanied by urgent appeals to Washington that the works under construction be built in the most permanent manner regardless of cost; that the reservoir dams be raised; the capacity increased; the canals made larger; and ditches built to every farm.

The third chapter opens with the practical completion of the works, the turning of the water to the fields and the beginning of the time of repayment. Then comes the condition predicted by former Speaker Cannon and other experienced politicians who fought the Reclamation Act. Their opposition was based frankly on the belief that when the time came to make payment to the Government the land owners would repudiate the claims. This opinion was based on a practical knowledge of human nature. These men were sure that the Federal Government could not collect from its citizens payments deferred over any considerable length of time. They had learned by experience that the only way for the Government to enforce payments is to have the cash in advance. The Post Office wisely demands payment before the mail is delivered; the Land Office, in contrast, shows a long list of unpaid claims, many of which have been subsequently cancelled by Congress.

While these statesmen (that is, politicians who have departed this life) clearly foresaw the ultimate conclusion, they did not point out, and, in fact, nobody appreciated, just how this amelioration of debts (not to use the harsh term "repudiation") might take place. Least of all did the engineers look forward to the treatment that was to be handed to them in the efforts to make them the scapegoats. No sooner were the terms of payment announced, however, than the same men, (the local land owners, bankers, and politicians, who had been urging that the engineers be instructed to do more work and make larger expenditures) reversed their position. Then began the long and heartbreaking campaign of attack which resulted in driving from the public service many able and conscientious engineers, technical assistants, men and women.

This is not the time nor the place to discuss matters of human and ethical relations, but they have peculiar interest to engineers in public employ, illustrating the fact that the moods of the public must be taken into account and studied with the same degree of care and impersonal attention as problems of fair and stormy weather, or drought and floods, which may destroy engineering works. Equal or greater destruction can come to the engineer and his organization through changes of public sentiment.

The doubts, censure, and personal abuse heaped on the engineers following the attempt to secure repayment from the land owners was naturally accompanied by investigations. No one has attempted to keep an accurate record of them, but the Fact Finding Commission has referred to several hundred, big and little. Some were notably expensive, not merely in the time and money involved in digging into every detail, but in distracting the attention of men in executive charge during times of the greatest stress, when every effort should have been concentrated on efficient work. It is literally true that men in direct charge of Federal reclamation projects, as well as their principal assistants, have worked night after night through long periods, assembling and re-assembling data called for. Under the strain of continued attack many a good engineer has broken, or has lost his temper at what to him has appeared the most ridiculous of accusations and demands. For example, one prominent Western Senator (now deceased) called on the Department for a list of all foremen employed on the works. No explanation was given; nor was the term "foremen" defined. At that time there were several thousand names on the payrolls, some designated as foremen mechanics, foremen carpenters, and every kind of similar designation currently used by contractors. The assembling of pay rolls and the careful scrutiny to distinguish between men who might be considered as foremen, required notable labor at a time when every effort should have been concentrated on keeping ahead of the floods and the demands for water. It is doubtful whether the Senator who asked for it even looked at the list.

This incident merely illustrates the conditions under which the Governmental forces have been compelled to work since the time the land owners were required to begin to make payment. It also illustrates the author's point that it is important to sever politics from business in the administration of this Governmental subsidy.

The question at issue is not as to whether a subsidy, as such, is a good or bad thing, but as to the misunderstanding which surrounds it and the importance of clearly bringing out, at all times, the amount and character of it. There is need here for unqualified clearness of statement; it is where the politician is most solicitous and most clever in concealing the facts. He is able to confuse the issue by a somewhat clever juggling of words. He uses the word "farmer" on all possible occasions, stressing the hardships and sufferings of the pioneer, and then the public is generally willing to expend its money freely.

Bear in mind the fact that the Government, in carrying out its reclamation policies, is dealing not with farmers as such, but with land owners. These land owners may or may not be farmers in the true sense of the word. The fact that they claim to own a piece of land which may be farmed, does not make them farmers. Possibly 40% of these land owners are non-resident. The hearings before the House Committee on Irrigation have shown that on the Government projects—to a greater extent than on private projects—the claimants to land ownership range down the alphabet from actors, bankers, and carpenters to undertakers, veterinarians, watchmen, and zoologists.

Every occupation is represented; unfortunately, there is not a preponderating percentage of men who would qualify under a strict test as real "honest-to-goodness" dirt farmers. Such men are usually too wise to undertake this kind of pioneering. They prefer to wait until the hardship and the uncertainties of the life of a pioneer have past, knowing full well that, when true land values are established, they can buy a good farm, carefully selected with reference to climate, crops, and market, practically at less than real cost.

The men who seek to enter on public lands that may be reclaimed, are necessarily adventurous and certainly optimistic and imaginative. They are not held back by the dictates of experience and, as a rule, have little anticipation of remaining long in the new country. They are usually attracted by the hope of getting something for nothing and of realizing the pioneer profits. This is no reflection on them. In fact, they are what are called "typical Americans", willing to take a chance, risking comfort and even the health of their families and themselves, looking forward to something better; each doing his part according to his ability.

The turnover in ownership is large. It is popularly stated that there are at least three successive owners before a relatively permanent home is established. This means in the aggregate large individual losses with occasional gains. Each successive owner has usually contributed something in the way of buildings and labor, more or less intelligently applied, adding to the real or productive values thereby. Sometimes, however, by careless handling of the water, the land itself has deteriorated, or become impregnated with salt or alkali; about 20% of the land has thus been injured.

The popular remedy proposed is that which occurs at first to every one, namely, that the ordinary run of citizens should not be permitted to exercise their individual discretion, but should be required to pass careful scrutiny before being permitted to enter on public land reclaimed or reclaimable by Federal funds. This is fine. As a practical thing, however, there are not enough applicants or purchasers to permit this kind of selection. It has been tried at various times and under various conditions without notable success. In fact, it comes down to this: That any man who has the faith and courage to enter on such an undertaking is more than welcome, and his shortcomings are excused. Moreover, experience with thousands of families shows that sometimes success has been made by the most unpromising specimens and, on the other hand, deplorable failures have been made by men who have come on the project with plenty of money and all the qualifications specified in an approved rule of procedure. It is the intangible, the mental attitude, a thing which cannot be measured by ordinary standards, that has resulted in success or failure.

While these changes in land ownership are taking place, on a Government project as contrasted with a private enterprise, there is a steadily increasing pressure for relief. Every one is heavily in debt, the bankers, to a certain extent, are desperate over the outlook; the local politicians advertise in speeches and in broadsides that if elected they will work for relief to the land owner. They are elected on this platform. As a result Congress, year

after year, has granted substantial relief. It is true that this is not the kind of repudiation predicted by former Speaker Cannon and others, but the effect is similar in that the whole amount of money advanced by the Government is not repaid. It becomes in effect what Mr. Henny describes as a subsidy and one that is partly necessary. It is not a matter to be grieved about, nor ignored; but fairness to the taxpayers of the country demands that the extent of it should not be concealed; nor should it be forgotten that private enterprise, which has reclaimed ten times the area of Federal enterprises, has had no such relief.

At present, it is peculiarly important that the public be taken into full confidence on this matter. The time is arriving when large claims on the public purse will be made for further engineering works in connection with reclamation. The Colorado River interests are demanding not millions but tens of millions of dollars. The land owners in the Columbia Basin are well organized in their demand for hundreds of millions. In fact, it is now the fashion to speak in terms of hundreds of millions when dealing with the subject of water control and development.

The cost per acre has caused several great reclamation projects to be rejected again and again; but they are still coming forward with ingenious schemes for repayment of costs (otherwise prohibitive) on the basis of a small percentage continued through several generations. Incidentally, of course, few seriously believe that Congress will ultimately insist on these large repayments, but will follow the well-established precedent of passing annual relief bills.

On top of these demands from the West, for the construction of large irrigation works with which to reclaim desert land, there comes the still greater and more urgent demand for the entire Mississippi Valley for immediate expenditure of hundreds of millions, not to extend the agricultural area, but to save that which has already become highly developed. Then to add to the demands of the people upon Congress, is the unsolved problem of the St. Lawrence River; the coastal or Atlantic inland waterways; the future of the Rio Grande; and the great host of flood protection, hydro-electric developments, such as Muscle Shoals, Great Falls on the Potomac, and other schemes stimulated in every part of the country. The advocates of all these are earnest, enthusiastic, and effective. The lobbies in Washington have never been so well organized and well financed. Lobbying, as such, is recognized as a necessity; even "an essential industry", in that it is impossible for the great body of Congress to understand the problems involved.

Engineers now have an opportunity to turn their attention, for a time at least, to these financial matters (pointed out by Mr. Henny) so that they will not be uninformed or swept off their feet and that the economists may be able to perform a larger notable public service by weighing the costs and benefits of the many great enterprises presented for public approval.

J. C. STEVENS,* M. AM. SOC. C. E. (by letter).†—This paper presents, in condensed form and in a very sane and rational manner, the present status of

* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.
† Received by the Secretary, August 13, 1927.

irrigation institutions in the United States. It is a sort of "looking backward" on the past 50 or more years of irrigation successes and failures.

One outstanding fact about the work of the U. S. Bureau of Reclamation deserves emphasis. To June 30, 1925, twenty-three years after the passage of the Reclamation Act, more than 82% of all monies due for construction charges and nearly 81% of all maintenance and operation charges had been paid into the United States Treasury by the settlers on the various Government projects.

When one considers that much of this work was pioneering in nature, often handicapped by lack of physical data, and hindered by political maneuvers, he must concede to Government reclamation a measure of success, equal if not superior to most corporate and private enterprises.

Another feature the author could well have included in his summary is that sound financing demands that all elements of cost be studied in advance. If drainage is essential, and it frequently is, it must be included. If settlers are to be furnished with land that has been leveled, ditched, and fenced; upon which houses have been built and implements and stock supplied; if expenditures are contemplated for colonizing; if the project must be "carried" until it is sufficiently settled to pay operation costs; these items must be included in the budget of the developing organization.

If the settlers are known in advance to be unable to meet all the items of cost, then their probable payments must be fairly estimated and the balance considered in the nature of a subsidy or carried over into an account to be amortized at some future time when the project shall have become a well ordered going concern. Many of the failures may be traced to an unwillingness on the part of the developing agency to reckon with or even inquire into these extraneous elements of cost.

The term, subsidy, is too often taken to mean cash payments, whereas it really means any aid to an enterprise for public convenience. All Federal Reclamation projects are subsidized to the extent that no interest is charged on deferred payments for construction costs. In general, such subsidies are more than repaid by the general increase in taxable values resulting from conversion of raw to productive lands.

The author's Conclusion (*d*)* that "the irrigation district method is not well adapted to the reclamation of large areas of desert land", must be considered in the light of the text of the paper regarding Class B lands the initial value of which is less than the funds that must be provided for reclamation.

While it is true that in the aggregate large areas of raw land distributed over a multitude of projects, have been successfully reclaimed by the irrigation district method, it is doubtful whether this method would have been successful had all these projects been thrown into one.

Settlers have diverse individual tastes and desires, and no one project or one locality will suit all. It is practically certain, therefore, that the colonization of a multitude of smaller projects, scattered throughout the West, has generally proceeded much more rapidly than if these areas were concentrated.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 912.

The success of any enterprise for the reclamation of raw lands hinges finally on the ability to colonize it to a point where interest and operating charges may be met. Failing this the "carrying costs" will wreck almost any private or district organization. They could only be met by Federal funds the use of which would thus constitute a form of subsidy.

The writer thinks the author could well modify his Conclusion (*d*), previously mentioned, to read: "The irrigation district method is not well adapted to the reclamation of large projects of desert land".

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STATE RECLAMATION IN WASHINGTON

Discussion*

BY F. H. NEWELL, M. AM. SOC. C. E.

F. H. NEWELL,† M. AM. SOC. C. E. (by letter).‡—This paper has peculiar value at the present time, particularly when read in connection with the current paper by D. C. Henny, M. Am. Soc. C. E., on "Some Phases of Irrigation Finance".§ The whole subject of the duties and opportunities of the National and State Governments in connection with water control, development, and protection of agricultural lands is forced upon public attention as never before by the great flood in the Mississippi Valley, which, although not necessarily the greatest that may occur, surpassed in destruction all previous floods.

Members of the next Congress (1927-28) are being urged, not only to control the Mississippi floods, but also to provide funds for building a storage reservoir to hold Colorado River floods. This is for furnishing electrical energy with which to pump water over a 1 300-ft. divide for Los Angeles, Calif., and neighboring cities. Claims are also being pushed for: Deep-water navigation in Northern New York; gigantic hydro-electric development on the St. Lawrence River; continued activity on the Atlantic deeper waterways; development of the international control of waters of the Rio Grande; the Columbia Basin project; and innumerable local flood-protection, irrigation, and drainage operations. The ten years' discussion of Muscle Shoals and of power development on the Potomac River near Washington, D. C., will continue to be the subject of active controversies on the floors of Congress.

There never was a time when water resources, water power, and regulation assumed such great public importance or involved expenditure of such huge sums from the unexpended balance in the Federal Treasury.

* Discussion on the paper by R. K. Tiffany, M. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Pres., The Research Service, Inc., Washington, D. C.

‡ Received by the Secretary, July 22, 1927.

§ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 896.

In this connection let it be recalled that the Federal Government itself, or at least the Constitution under it, which has operated for nearly a century and a half, originated through a similar set of demands arising from the necessity of protecting commerce between the States. At that time commerce was largely dependent on waterways. The Constitutional Convention was called partly because of the controversies between the States over the control of the rivers and the obvious necessity of creating one central government strong enough to adjust the conflicting interests.

At present, there is a demand for the development of a sound policy on the relation of the States to each other and to the Nation. The relative duties of each concerning water and the distribution of power across State boundaries are involved in this question.

The engineer is at the center of the picture. While the politician necessarily strives to attract attention to the wishes of his constituents, he realizes that he must have dependable facts. He may denounce engineers but he displays an almost childlike faith in the facts and figures issued by them. It is, therefore, peculiarly important at the present time for the engineers of the United States to present, as clearly as possible, the fundamental facts on which proposed appropriations and accompanying legislation must be based. These appropriations are no longer discussed in terms of millions of dollars; but, with the boldness, not to say recklessness, of expenditure growing out of the World War, the talk is in terms of hundreds of millions of dollars; amounts so large that even the wealthy States must call upon the Federal Treasury for aid.

Back of all these questions of reclamation and other developments comes the inquiry, more and more insistently made, "Will it pay?" A quarter of a century ago, when the Reclamation Act of June 17, 1902, was passed, this question was not asked. The enterprises then proposed were relatively cheap. There were a score of applicants for every farm that might be irrigated or drained.

In the quarter of a century that has passed there has been a reversal. All the relatively cheap projects have been undertaken or have passed under private control. There are no longer hordes of eager land seekers. On the contrary, thousands of people are leaving the farms each year; millions of acres of land, previously cultivated, are now left untilled. While it is popularly deplored, this condition is perfectly logical.

For years, the Federal and State Departments of Agriculture have employed the best experts obtainable. They have spent millions of dollars trying to find how to produce larger and better crops on less area and with less outlay of human energy. They have succeeded in their aims. Thousands of laborers have been released for work at more profitable wages in industry. The scientists have brought about the introduction of better methods of farming, better judgment of soils, and have encouraged, indirectly at least, the concentration of efforts by good farmers, on good soil, near good markets. The resulting suffering of poorer farmers on poorer lands and without markets is deplorable, but is the inevitable outcome of conditions that have been deliberately planned.

In the face of the rapid and, in some cases desirable, decrease of area cultivated, is it wise for the tax-payers of the Nation or of a particular State to be called upon to reclaim additional areas, even if the soil may be brought to a high degree of productiveness by large expenditures? After that comes the question as to who is to pay the cost of strenuous efforts to secure good farmers and good markets?

The various efforts of the State of Washington, made in good faith, to develop lands and to secure settlers are described by Mr. Tiffany. He shows that the early individual or co-operative efforts at irrigation were successful. The pioneers, counting the cost of their labor as nothing, succeeded in making homes and, in some cases, developed lands of high productivity. Then having "skimmed the cream", they began to fail in further efforts. Later, the State endeavored to aid or stimulate new projects. Many were investigated; considerable sums of money were spent and lost. Now the State has apparently settled down to an effort, which may be characterized as the salvage of enterprises, that offers hope of rehabilitation or extension.

Through or behind these efforts runs the fundamental thought, unquestioned up to the present, namely, that eventually the State of Washington (and, indeed, each and every State) "will need all possible development of her agricultural area". Accepting this as a fact, broadly applicable to all industries and human undertakings, the real question then resolves itself into what shall be included as possible.

By offering suitable inducements, it is possible to put people on the reclaimed land who will agree to cultivate it; but is it worth while? The experience of the State of Washington, as described by the author, appears to be in the negative. It has not paid, and it is only by the exercise of supreme faith and optimism that the expenditure of hundreds of millions of dollars additional may be justified.

This does not mean that extension, enlargement, and improvement of existing systems is unprofitable or unwise. Much can and will be done, especially by local interests receiving an indirect benefit. Many land owners, who now have a "toe-hold" on the soil, and disregard the value of their own labor (which otherwise might be dissipated), can, and will, join with their neighbors and bring about gradual expansion of irrigated area. This expansion is taking place throughout the arid region, often by the formation of self-taxing reclamation districts. These districts are not held strictly as to questions of profit and loss. They are justified in levying taxes on the basis of the general benefit that came, directly or indirectly, to all industries or to commercial enterprises in the neighborhood.

These relatively small neighborhood undertakings will take care of themselves without other supervision than that generally given to school districts. However, there are the great enterprises like the Columbia Basin project, involving enormous expenditures, with potential losses in interest and capital, before any lands are reclaimed; investments too great for even a wealthy State to make.

In these great undertakings the engineering features, the methods and costs are of course, fundamental. In addition, there is the less easily defin-

able question as to how the improved land may be utilized so as to return a considerable part of the total cost. Theoretically, it is possible for the State and Federal Governments to join hands in securing settlers; but in view of the fact that all such undertakings in the past have been at least doubtful, it is now urged that the Federal Government buy up or acquire all the land that may be reclaimed.

As the owner of the reclaimed lands, the Government may be able to dictate to its tenants or purchasers the conditions under which the lands may be acquired. It can enforce needful conditions of cultivation to a degree comparable to that of the owner of a manufacturing establishment. Recital of these conditions cause strong doubts as to their feasibility. The successful farmer is a man who is called on to use his judgment hour by hour, his activities are constantly being modified by changes in weather, or by other conditions known only to himself. Judgment is essential, and if he surrenders his judgment to some one else, he loses those qualities that make him a real farmer.

It seems little short of a huge joke to imagine any bureau officer, selected in accord with Civil Service rules and made an expert farmer by virtue of a commission, undertaking to direct the activities of a body of real farmers. The earnings of any farmer should be larger than the salary usually paid to subordinate Federal employees. Imagine the responsibility assumed in directing a group of farmers; what crops to plant and when to harvest. Perhaps the potatoes might be successful, but if not sold at a profit who is responsible? In the opinion of the farmer the Government should at least make good for any such loss.

If, as is intimated, the Federal Government should come to the assistance of the State of Washington, in building the Columbia Basin project, what about the other States? Each and all of them from Maine to Florida in the East, to California and Washington in the West, has a comparable problem, of water control, water power, drainage, or irrigation. Each and all argue that the agricultural possibilities should be maintained and water controlled or developed for power or other uses. Each State pleads that although the wealth per capita is great it cannot afford to go into these great enterprises; each is equally insistent that if the Federal Government assists one State it should also consider the needs of the other.

It has already been proposed with some seriousness, that if money is spent on the arid soils of the West, similar funds should be used in clearing the rocks and stumps from the otherwise productive farms in the East. It is pointed out that with such Federal aid and with the existence of the best markets in the world, the Italians, Portuguese, and other prolific immigrants in New England can, and will, develop an agriculture comparable to that on the irrigated lands of the West. As engineering enterprises, and as economic investments, both proposals are claimed to be equally justifiable and altogether as valuable to local industries.

All engineers and economists can agree on the program outlined by the author: namely, that a comprehensive study be made of the possibilities and need of the State for increased agricultural production; that continued effort

be made to improve State laws relating to reclamation; and that the State should support a National reclamation policy and program based on sound economic principles.

It is in the development of such sound economic principles that there has been and still remains the greatest need. In a certain sense, this country has been limping along under assumptions that were true a quarter of a century ago and do not apply to present conditions. The needs and opportunities of the West then were unquestioned. It was assumed that it was the duty of the Federal Government to give priority to everything demanded by the West as the child or ward of the Nation. The West, however, has advanced, and a broad review of conditions shows that in many respects it has outstripped the remainder of the country, particularly the South.

Southern statesmen, under the leadership of the late Francis G. Newlands, Senator from Nevada (but a native of Mississippi), have consistently supported the claims of the West. Since the death of Senator Newlands they have begun to adopt a more questioning attitude. In fact, several of them have definitely stated to their constituents, and particularly to the engineers and economists of the South, that if Federal aid is to be given anywhere, the South, with its pressing needs for flood protection, must have preference. Southern interests are studying the methods so successfully developed by the West. They have been in contact with the competent men sent from the State of Washington to urge appropriations for Yakima Valley and Columbia Basin. They are awaking to the fact that their own States have enterprises of equal or greater importance. It is quite possible that in succeeding sessions of Congress the engineers representing these enterprises in the South will be fully as effective and energetic as those from the West.

The discussion of methods and of Federal and State responsibilities in water control is thus coming down to hard engineering facts. The relative costs and values and the real responsibilities are being seriously studied. The tax-payers are slowly realizing the fact that these subsidies, which they are called on to pay for the benefit of land owners in distant States, do not always produce the desired results. The emotional attitude, which has been so successful in securing contributions or appropriations for relief, is giving way to questions that can be answered only by engineers who will discuss the costs and benefits impartially and clearly.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS WILLIAMS BALDWIN, M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1926.

Thomas Williams Baldwin, the son of Thomas Williams and Margaret Josephine (Bacon) Baldwin, was born on December 27, 1849, at Bangor, Me., where his parents were then residing.

He was a descendant on his mother's side of Stephen Hopkins who came to America on the *Mayflower* and on his father's side of Thomas Cushing who was a member of the Continental Congress, as well as Acting Governor of Massachusetts in 1785, and of John Avery, Secretary of State in Massachusetts for several terms in Revolutionary times.

Mr. Baldwin prepared for college at Phillips Academy, Exeter, N. H., from which he was graduated in 1869. He was graduated at Harvard College in 1873, and, in 1874, entered the Junior Class of the Massachusetts Institute of Technology, from which he was graduated in 1876 with the degree of Bachelor of Science. During vacations in his college years he served as Rodman and Transitman on surveys for the European and North American, and the Bangor and Piscataquis Railroads.

In 1878, Mr. Baldwin opened an office in Bangor, Me., for the practice of his profession, and was Acting City Engineer until March, 1879, when he was elected City Engineer, which position he held until 1883. Declining further service in that office he accepted the position of Division Engineer on the Bangor and Piscataquis Railroad extension work, but retired after a few months' service on account of illness. Upon recovery he returned to private practice in his Bangor Office.

In 1895, he removed to Boothbay Harbor, Me., to engage in business and, in 1903, became President of the Baldwin Fish Company, which position he held until he removed to Boston, Mass., in 1908.

While in Boothbay Harbor Mr. Baldwin served for ten years on the School Committee, and was a Representative for two terms in the Maine Legislature. He was also a Director of the First National Bank and a Corporator of the Savings Bank of Boothbay Harbor.

On removing to Massachusetts, he became actively interested in genealogical research and published a genealogy of William Patten, who, in 1636, settled in Cambridge, Mass.; also one of Michael Bacon who was in Natick, Mass., in 1640. In addition, he edited and published with a memoir the "Revolutionary Journal" of Col. Jeduthan Baldwin, his great-grandfather.

* Memoir prepared by John Carroll Chase, M. Am. Soc. C. E.

He also compiled and published the vital records of twenty-one Massachusetts cities and towns, a lasting monument to his patient and persevering labors.

Mr. Baldwin held minor positions with the Highway and Civil Service Commissions of Massachusetts, after taking up his work of compiling vital records, but the state of his health did not permit active practice of his profession.

Although of a retiring disposition he always had a keen interest in people and in public events; and was patriotic, affectionate, and loyal. His liking for historical and genealogical matters made his later occupation very congenial to his tastes. In his more active years, he had traveled extensively throughout the United States and Canada.

Mr. Baldwin was a Life Member of the American Unitarian Association and was affiliated with the Republican party. He was also a member of the New England Historic Genealogical Society, the Sons of the American Revolution, and the Society of the Cincinnati.

After a short residence in Boston he lived in Wellesley, Cambridge, and Hardwick, Mass., where the end came after several years of failing health.

On December 8, 1880, he was married to Maud, daughter of John and Emeline (Young) Patten, of Bangor, who survives him, with two daughters, Mrs. Chalmers Stevens Clapp, of Abington, Mass., and Miss Dorothy Arno Baldwin, of Hardwick. He also leaves four grandchildren.

Mr. Baldwin was elected a Member of the American Society of Civil Engineers on April 2, 1884.

JOSEPH WRAY HUNTER, M. Am. Soc. C. E.*

DIED MAY 19, 1927.

Joseph Wray Hunter was born on July 23, 1853, in Haverford Township, Delaware County, Pennsylvania. His father was Thomas and his mother, Henrietta (Schwend) Hunter. His early life was spent in Lower Merion Township, Montgomery County, and Radnor Township, Delaware County, Pennsylvania, and he was educated in public and private schools, having been graduated in 1870, second in his class from the Mantua Academy, in West Philadelphia, Pa.

Mr. Hunter's first work was with the late Samuel L. Smedley, M. Am. Soc. C. E., Surveyor and Regulator of the 11th Survey District, of Philadelphia, with whom he served until 1874, when he became engaged with Mr. G. M. Hopkins on map survey work. In 1875, he moved to Jenkintown, Pa., where he resided for the remainder of his life. In 1878, he engaged in the private practice of surveying in Jenkintown, and, in the same year, entered public life by being elected Justice of the Peace in his community.

Mr. Hunter started his long experience in highway work when, in 1890, he began, as the Borough Engineer of Jenkintown and Township Engineer

* Memoir prepared by H. A. Thomson, Assoc. M. Am. Soc. C. E.

for Cheltenham and Abingdon Townships, the construction of roads of the macadam and Telford macadam types. From 1902 to 1904 he was a member of the State Commission appointed to determine, survey, and re-establish the boundary line between the City of Philadelphia and Delaware County, Pennsylvania. He carried out a considerable part of the field work in this connection.

In 1903, he was appointed by Governor Pennypacker as the first State Highway Commissioner of Pennsylvania. He organized this Department and served in it in various capacities until his death.

During his early life Mr. Hunter filled a number of positions to which he was elected and in which he served faithfully, as he held all such matters of public trust in the highest regard. He was elected and re-elected Justice of the Peace. He was elected County Surveyor of Montgomery County, in 1872, and in 1887 Register of Wills for the same county. These elections and the manner in which he filled the positions led to his appointment as the first State Highway Commissioner. When he appointed Mr. Hunter to this position, Governor Pennypacker expressed his high regard for him as an Engineer, and this expression was repeated by successive Governors who re-appointed him to the Department. He served under Governors Pennypacker, Stuart, Tener, Sproul, Brumbaugh, Pinchot, and Fisher.

Mr. Hunter was outstanding as an engineer, a public servant, and, most of all, as a man. He was the most widely known and most loved man in highway circles in Pennsylvania. He had a depth of understanding and sympathy that won the unswerving loyalty of hundreds of young men whom he started on careers and whose close friendship he retained to the end. He was widely known to thousands of Township Supervisors of Pennsylvania whose counselor he was for many years and whose State Association had conferred on him the honorary title of "Dean of the Highway Department."

To thousands of these Supervisors as well as to hundreds of engineers in the Highway Department the name of "Uncle Joe" Hunter was revered. His advice was sought and heeded by many. He was a man who knew highway construction and was ahead of his time in the early years of the State Highway Department. He had tact and judgment; he did not force his ideas on an unwilling populace. He made suggestions quietly and lived to see his plans successfully adopted.

Mr. Hunter was closely associated at all times with the men who built the roads as well as with the people who provided the funds with which they were built. In laying the foundations for the great highway system of Pennsylvania, he had to deal with what was at first an unwilling majority, but by persistently demonstrating that his ideas were correct, he succeeded in converting his opponents to his views. This he accomplished not only by the soundness of his engineering ideas, but by his sense of appreciation of the other man's viewpoint. His was not a single-track mind, nor did he ever seek publicity. He was a mild-mannered, kind-hearted gentleman.

Mr. Hunter had membership in a number of technical clubs and societies, having been active in the American Society for Testing Materials and the

Atlantic Deeper Waterways Association. He was a member of the American Road Builders Association; the Engineers Club of Philadelphia; Friendship Lodge, F. and A. M., of Jenkintown; Lu Lu Temple, A. A. O. N. M. S., of Philadelphia, and other branches of the Masonic order.

As long as the Pennsylvania State Department of Highways exists no history can be complete without a full and grateful recognition of the services of Joseph Wray Hunter in laying the foundations for the great organization in its early days. He will be remembered particularly for his services in organizing the Township Division and in purchasing the private turnpike system of the State, but most of all for his kindly and valuable advice given to successive executives of the State Highway Department. He was indeed the type of man for whom many treasure the thought that "he was my friend."

Mr. Hunter was elected a Member of the American Society of Civil Engineers on May 3, 1910.

WILBER MACAULAY JUDD, M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1924.

Wilber Macaulay Judd, the son of the Rev. Frederick F. Judd and Jeanette (Wilber) Judd, was born at Parsippany, N. J., on June 20, 1862. He entered Union College, Schenectady, N. Y., for the regular civil engineering course and additional academic subjects, so that at his graduation in 1884 he received both the degrees of Civil Engineer and Bachelor of Arts. His standing was the highest in his class, entitling him by appointment of the Faculty to the highest commencement honor, that of Valedictorian.

His first engineering work was done under the direction of his college preceptor, Cady Staley, M. Am. Soc. C. E., on the construction of a sanitary sewerage system for Schenectady. This employment led to his becoming associated with the firm of Staley and Pierson in sanitary and hydraulic engineering work, which connection was continued during the first fifteen years of his professional career.

Some of the important pieces of work of which Mr. Judd had charge during this period were, as follows: From 1886 to 1887, he was Assistant City Engineer of Kalamazoo, Mich., and from 1887 to 1888, he had general charge of the construction of the sewerage system of Watervliet, N. Y. From 1890 to 1891 he served as Assistant City Engineer of Lincoln, Nebr., and from 1891 to 1894 he was in general charge of the construction of sanitary sewerage systems for Dayton and Conneaut, Ohio. From 1895 to 1896 he held the position of Assistant City Engineer of Peoria, Ill., and in 1897 was Construction Engineer for the Merodocia Levee and Drainage District, Albany, Ill. During 1898 and 1899, he carried out all civil engineering work for the Illinois Steel Company, at Joliet, Ill.

* Memoir prepared by Professor James H. Stoller, Dept. of Geology, Union College, Schenectady, N. Y.

In 1899, Mr. Judd became associated with the late Selwyn M. Taylor, M. Am. Soc. C. E., and was in charge of mine development, etc., at Luzerne, Pa. Later, he was Resident Engineer for the plants at Luzerne, Footedale, and Buffington, Pa.

In 1903, he became a member of the firm, The W. G. Wilkins Company, Engineers and Architects, of Pittsburgh, Pa., and continued in this partnership until his death. As a member of the firm Mr. Judd had general charge of drafting-room work, estimating, and designing, and more or less field supervision of all civil engineering and mining work.

As to his standing as an engineer the writer quotes from a letter written by George S. Pierson, M. Am. Soc. C. E., with whom Mr. Judd was long associated:

"Mr. Judd was exceptionally well grounded in his profession. His personality was most pleasing and his judgment excellent and of a quality of expression and fairness as between conflicting interests that was most convincing."

Mr. Judd was married on April 9, 1917, to Marietta Fallsbush, of Pittsburgh, who survives him.

As known to his college classmates he was of a quiet and modest bearing, genial and companionable to those who sought him out, a man of clear, strong mind and fine character.

He was a member of the Engineers Society of Western Pennsylvania; the Coal Mining Institute of America; the Chamber of Commerce of Pittsburgh, and the Civic Club of Allegheny County, Pennsylvania. He was also a member of the Presbyterian Church.

Mr. Judd was elected a Member of the American Society of Civil Engineers on October 4, 1905.

CHARLES LESTER PARMELEE, M. Am. Soc. C. E.*

DIED APRIL 4, 1927.

Charles Lester Parmelee, the son of Myron Holly and Emma (Lester) Parmelee, and a descendant of one of the founders of Guilford, Conn., was born at Syracuse, N. Y., on January 16, 1873. He spent a year studying civil engineering at the University of Michigan and, in 1890, went to the Massachusetts Institute of Technology, from which he was graduated in 1895 as Bachelor of Science in Civil and in Sanitary Engineering.

His college vacations were spent in civil engineering work. From 1895 to 1898 he served as Assistant Engineer to George W. Fuller, M. Am. Soc. C. E., on the experimental water filters at Louisville, Ky., and again on the experimental filters and preliminary designs for the Cincinnati, Ohio, Water-Works.

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* Memoir prepared by Alfred D. Flinn, M. Am. Soc. C. E., and John B. Stein, Assoc. M. Am. Soc. C. E.

From 1898 to 1902, Mr. Parmelee was Chief Engineer, successively, of the Continental Filter Company and the New York Continental Jewell Filtration Company, in charge of design and construction of filtration works for many communities and industrial plants, including the works at Little Falls, N. J., the first large concrete filter plant built on the American system. For the next two years he acted as Consulting Engineer to the New York Continental Jewell Filtration Company on various enterprises, including a dry dock for the Federal Government, at Charleston, S. C., Oak Lane Reservoir, Philadelphia, Pa.; First Street Tunnel of the Pennsylvania Railroad Company, Washington, D. C.; Wantagh Infiltration Works of the Brooklyn, N. Y., Water Supply, and other work having an aggregate value of about $5 000 000.

In 1904 and 1905 he held the position of Consulting Engineer to the City of Toledo, Ohio, on water-works improvements, including a 32 000 000-gal. filter plant. In 1906 and 1907 he served as Consulting Engineer to J. G. White and Company, in connection with a dam across the Colorado River near Yuma, Ariz., preliminary studies for the Los Angeles Aqueduct, and other investigations.

From 1907 to 1912, Mr. Parmelee was Chief Engineer of the T. A. Gillespie Company, General Contractors, and the East Jersey Pipe Company, manufacturers of lock-bar and riveted steel pipe. These engagements connected him with contracts totaling approximately $15 000 000, including water purification works for Pittsburgh, Pa.; two locks and dams on the Ohio River; the Rondout Creek and Hudson River pressure tunnels of the Catskill Aqueduct, of the New York, N. Y., Water Supply; Granite Dam across the Yadkin River, at Whitney, N. C.; extension of the power plant at Massena, N. Y.; pipe lines at Brooklyn and Lockport, N. Y., Pittsburgh, Pa., and Portland, Ore.

For the next six years he held the position of Consulting Engineer to a number of contractors on large construction in hydraulic and railroad undertakings. He also made examinations of railroad, water-power, water-works, timber, oil, gas, and coal projects. In 1917, Mr. Parmelee served as Captain in the Engineers Reserve Corps, on cantonment work, and in 1917 and 1918 as Consulting Engineer for the Oil Division, United States Fuel Administration. From 1918 until his death, he practiced as a Consulting Engineer, mainly in charge of the construction of oil refineries for the Sinclair Refining Company, aggregating $40 000 000 in value.

In 1896, Mr. Parmelee was married to Helen F. Donnelly, who died in 1920. Two daughters by this marriage, Emma Lester and Helen Donnelly, survive him. In 1923, he was married to Gertrude M. Pankan, who also survives him.

He was a member of the following societies: American Institute of Mining and Metallurgical Engineers; The Engineers' Club of New York; Railroad Club of New York; University Club of Chicago, Ill.; Essex County Country Club of West Orange, N. J.; Glen View Club of Illinois; and Old Colony Club of New York.

Mr. Parmelee was elected a Member of the American Society of Civil Engineers on February 4, 1913.

THOMAS WAKEFIELD MACARTNEY, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 23, 1926

Thomas Wakefield Macartney was born in Des Moines, Iowa, on November 20, 1883. Following his preparatory education in the public schools of Des Moines, Mr. Macartney received his technical and engineering training at Purdue University from which he was graduated with the degree of Bachelor of Science in Mechanical Engineering in 1906.

Soon after his graduation he accepted a position as Draftsman with Mr. W. S. Hook at Los Angeles, Calif. After serving a year in this position he entered the office of the City Engineer of Seattle, Wash., where he served as Computer, Draftsman, and, later, assisted in the preparation of plans for street improvements, sewers, and extensions to the water system. He was also connected for one year with the Municipal Plans Commission.

In January, 1914, Mr. Macartney left Seattle to accept a position as Assistant County Drainage Engineer in Yakima County, Washington. In this position he assisted in the location, design, and construction of many large drainage systems and acquired much valuable experience in this field of engineering.

In August, 1917, he left the service of Yakima County and entered the employ of the Boering Airplane Company, of Seattle, soon afterward becoming its Chief Inspector.

He returned to Yakima County in January, 1919, as Chief Assistant Drainage Engineer and was appointed Chief Drainage Engineer in August, 1919, which position he held until his death. During this period he had charge of the design and construction of thirty drainage improvement districts having an aggregate area of 25 000 acres. On this work 20 miles of large open drains and 60 miles of deep closed drains were built at a cost of $876 000. The construction was difficult for this type of work and much of it was done by force account under the direction of Mr. Macartney. In addition, he had charge of the design and construction of several diking improvement districts for bank protection on the Yakima River.

In the field of drainage engineering for the reclamation of irrigated land, Mr. Macartney stood very high, indeed, ranking with the best. The drainage systems that were built by him are models of excellent design and good construction. They reflect the skill and ability of the builder and the Yakima Valley owes to him a debt of gratitude for his work of reclamation and protection.

Mr. Macartney accepted responsibility conscientiously and served the public with highest integrity. His fine tact, good judgment, and even temperament enabled him to work harmoniously with many different district boards and commissioners. He was constantly and steadfastly guided by the best principles of engineering and sought to do that which was for the

* Memoir prepared by L. T. Jessup, Assoc. M. Am. Soc. C. E.

greatest good. He was wholly without conceit or sham, and his courtesy and lovable disposition endeared him to all who knew him. His very sudden and untimely death caused by pneumonia was a severe shock to his many friends.

He took an active part in civic and community affairs. At the time of his death he had served eight years as Vestryman of St. Michael's Protestant Episcopal Church, at Yakima, Wash. He was a member of the Rotary Club and of the American Association of Engineers.

He was married on September 6, 1911, to Margaret Scarborough, by whom he is survived. He is also survived by a son, Thomas Wakefield, Jr., a sister, Catherine N. Macartney, of Iowa City, Iowa, and three brothers, George W. Macartney, of Helena, Mont.; Robert H. Macartney, of Cheney, Wash., and Morton Macartney, M. Am. Soc. C. E., of Orlando, Fla.

Mr. Macartney was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

RALPH JEROME SMITH, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 8, 1926.

Ralph Jerome Smith, the only son and youngest of five children of the late Charles Aaron and Jennie (Snow) Smith, was born at Indian Orchard, Mass., on May 12, 1889. Here he resided until he moved with his parents to Longmeadow, Mass., in 1912, which has since been the family home. He was the direct descendant of a long line of old New England families. Through his mother, he was descended from John Alden and Priscilla Mullens, also from Francis Cooke, all of whom were passengers on the historic *Mayflower*. His paternal New England ancestry, of which he was the last male member in his line, dates from 1636 when Lieut. Samuel Smith with his family removed from Hadleigh, England, to America on the ship *Elizabeth*, settling first at Watertown, Mass., later, at Wethersfield, Conn., and, finally, going with a party from that place to settle the Town of Hadley, Mass. Ralph Smith's great-great-grandfather, Ebenezer Smith, was a soldier in the Revolutionary War.

Ralph Jerome Smith received his early education at Indian Orchard, having attended the Myrtle Street Grammar School. He then entered the Springfield, Mass., High School with the idea of fitting himself for the Medical Profession. About a year and a half later his father was severely injured, and he was obliged to leave school for a time to look after his father's business. Upon the latter's recovery, the business was disposed of, and Mr. Smith then entered Williston Seminary. He found that his professional leanings had changed from medicine to engineering, so he returned to the Springfield Technical High School from which he was graduated in 1909.

* Memoir prepared by Carl C. Cooman, Assoc. M. Am. Soc. C. E., and Robert B. Jeffers, M. Am. Soc. C. E.

After leaving High School he worked as a Draftsman on machine detailing for the Consolidated Wrapping Machine Company, of Springfield. About a year later he became a Valve Draftsman for the Chapman Valve Manufacturing Company, of Indian Orchard. He then spent about two years as Rodman and Chainman on surveys in and around Springfield, first with Durkee, White, and Towne, of Springfield, then with the General Engineering Company of the same place, and, finally, with the Boston and Albany Railroad Company.

From 1912 to 1915, Mr. Smith was employed by the General Engineering Company, of Springfield, on reservoir and hydro-electric development work, road construction, and real estate development. During this time he worked as Computer of capacities of storage reservoirs; Inspector of wash-borings and soundings for the foundation of a proposed dam; Instrumentman on levels and surveys; Draftsman on maps and proposed buildings; Chief of Party on topographic survey for hydro-electric development; and Superintendent of road construction and general real estate development. He then spent a year as Production Clerk for the Strathmore Paper Company of Woronoco, Mass.

In 1916, he became associated with the Fred T. Ley Company, of Springfield and New York, N. Y., and for the next four and one-half years he was engaged in that connection on construction work in the eastern part of the United States. He spent a year as Local Purchasing Agent on the construction of the James Deering residence at Miami, Fla. He then became Engineer of Construction for a large group of farm buildings built by the Company for the owner of the McCallum Silk Mills, at Plainfield, Mass.

During the World War, Mr. Smith was Employment Manager for the Fred T. Ley Company, Contractors, on the construction of Camp Devens at Ayer, Mass., the Ammonium Nitrate Plant for the Atlas Powder Company at Perryville, Md., and the aerial bomb-loading plant for the Marlin-Rockwell Loading Corporation at Rockwell Park, Mount Pleasant, Del. After the signing of the Armistice, he was employed in adjusting accounts in dispute between the vendors and the Government Accounting Department on the construction of Camp Devens, and as Custodian of Property for the Contractor during the period of transfer of the buildings and materials from the Contractor to the Government. Returning to Springfield, he became Specification Writer and Personnel Manager in the Company's Engineering Department.

In 1919 he was sent to Rochester, N. Y., as Engineer and Superintendent of Construction for the Company during the construction of a factory building for Bastian Brothers Company of that place.

In October, 1920, Mr. Smith became Assistant Engineer with the Eastman Kodak Company, of Rochester, N. Y., and was employed for five years in the Engineering and Maintenance Department at the Kodak Park Works. He at first supervised building construction in the field and, later, had charge of the planning and scheduling of shop work for the Department.

His health had been poor for some time and for this reason, thinking that a change of climate would benefit him, he went to Fulford, Fla., near Miami,

in the fall of 1925, on real estate development work for Mr. H. B. Graves of Rochester. The climate, however, did not agree with him, so he returned to Rochester the following spring. On June 1, 1926, he began work as City Engineer of Olean, N. Y., which position he held at the time of his death.

Mr. Smith was an active member of the Masonic Fraternity. He was a life member of Brigham Lodge, A. F. and A. M., Ludlow, Mass., of which lodge his father was a Charter Member. He was also a member of the Morning Star Chapter, Royal Arch Masons, Springfield, Mass.; Springfield Council, R. and S. M.; Cyrene Commandry No. 37, Knights Templar, Rochester; Rochester Consistory, A. A. S. R., Rochester; Damascus Temple, A. A. O. N. M. S., Rochester, and Bela Grotto, M. O. V. P. E. R., Springfield. He was also a member of Lodge No. 948, Benevolent and Protective Order of Elks, of Miami.

From early childhood Mr. Smith was very fond of music, particularly of singing, and was possessed of a beautiful tenor voice. As a lad, he sang in the children's choir in the Evangelical Church at Indian Orchard. During his High School career, he was a member of the Glee Club. He also sang in the choir of St. Peter's Protestant Episcopal Church and, later, of Trinity Methodist Episcopal Church, at Springfield. During his stay at Perryville, Md., he sang in the Methodist Episcopal Church at Havre de Grace, Md., and while in Miami, in 1916, in the White Temple at tenor soloist.

When he went to Rochester in 1920, Mr. Smith transferred his church membership to the Central Presbyterian Church, where he sang in the choir. He was also a member of the Chanters of Damascus Temple at Rochester and of a Male Octette organized among the Kodak Park employees of the Eastman Kodak Company. He often appeared at special functions as a soloist and occasionally sang over the radio after its advent as a regular medium of entertainment.

During his Grammar School days, he once won a story-writing contest given for the school children of Springfield by the *Springfield Daily News*. The prize was a fine picture which still hangs on the walls of the Myrtle Street School bearing a plate inscribed with his name and the reason for the award. He never cultivated this natural bent for writing, but was always a great reader, being interested especially in books on science, religion, philosophy, and theosophy.

Mr. Smith was a tall, rather heavily built man, deliberate of movement, and possessing a quiet, cheerful, friendly nature. He impressed all with whom he came in contact as being dependable, honorable, and unassuming. Those with whom he had any dealings found him to be all that his appearance indicated. He had a marked ability of smoothing out difficulties in a calm, unhurried manner that was usually effective and, at the same time, convincing and reassuring to all concerned, whether in personal or professional association. He was in every respect a gentleman.

On November 6, 1920, Mr. Smith was married to Mary E. Decker, of Rochester, formerly of Coudersport, Pa., who survives him. There were no

children. He is also survived by his aged mother, Mrs. Jennie Snow Smith, and two sisters, Mrs. Frederick N. Fowler, and Mrs. Edson R. Dorman, both of Longmeadow, Mass.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on October 10, 1921. During his sojourn in Rochester he was active in the Rochester Local Section. He was also a member of the Rochester Engineering Society.

HARRY BRONSON SNELL, Assoc. M. Am. Soc. C. E.*

DIED JULY 3, 1927.

Harry Bronson Snell was born in New Haven, Conn., on March 28, 1875, the son of Adolphus Gaylord and Mary Andruss Snell.

After his graduation from Sheffield Scientific School, Yale University, New Haven, Conn., he became Assistant Civil Engineer with the Hartford Street Railway Company, until August, 1903. He then accepted the position of Assistant Engineer for the Long Island Railroad Company, Atlantic Avenue Improvement, Brooklyn, N. Y.

In September, 1905, Mr. Snell was appointed Principal Assistant Engineer for the Brooklyn Grade Crossing Commission, Brighton Beach Improvement, and, in 1916, he became Superintendent of Construction for the Turner Construction Company, of New York, N. Y., and Boston, Mass., which position he held until his death.

Mr. Snell died at the Griffin Hospital, Derby, Conn., on July 3, 1927, following an operation for appendicitis and after an illness of only a week. He had just attended the re-union of his class at Yale.

On February 2, 1916, he was married to Emily M. Sturdevant, of Deposit, N. Y., and is survived by his widow and by his brother, Thomas C. B. Snell, M. Am. Soc. C. E.

Mr. Snell was elected an Associate Member of the American Society of Civil Engineers on December 5, 1906. He was also a member of the Sons of the American Revolution.

ROBERT TEETERS SOMERS, Assoc. M. Am. Soc. C. E.†

DIED APRIL 5, 1927

Robert Teeters Somers was born in Angola, Ind., on September 21, 1889. He received his early education in the public schools of Auburn, and Huntington, Ind., and afterward attended Oberlin College where he received the degree

* Memoir compiled by T. C. B. Snell, M. Am. Soc. C. E.

† Memoir prepared by Irving D. Goodwin, Assoc. M. Am. Soc. C. E.

of Bachelor of Arts in 1914. The same year he entered Purdue University from which he was graduated in 1916 with the degree of Bachelor of Science in Civil Engineering.

Mr. Somers began his engineering career as Draftsman in the Pittsburgh, Pa., Office of the Pittsburgh-Des Moines Steel Company. Soon after the United States entered the World War he tried to enlist in the Coast Guard Division, but was rejected because of near-sightedness. About this time the Pittsburgh-Des Moines Steel Company began fabricating eight radio towers for the Lafayette Radio Station in France. Mr. Somers continued with this firm as Production Engineer until 1920. This work involved the adjustment of steel stock to the requirements of contracts and overseeing shop schedules of contracts.

Long hours of service during the war period, followed by an attack of influenza, undermined his health, and from February, 1920, until June, 1921, he lived in the mountains of Pennsylvania.

After his recovery, he assisted Mr. E. Paul Rothrock, who is now State Geologist of South Dakota and Head of the Geology Department of the University of South Dakota, in plane-table mapping of the topography and structure in Lawrence County, Kentucky, for the Washington Oil Company, of Pittsburgh. On the basis of this survey, the Company leased considerable land.

When this work was completed Mr. Somers went temporarily with the Pittsburgh Division of the United States Bureau of Standards, his work consisting of testing the strength of certain materials and computing the strain which would bring the breaking point.

When the Bureau of Standards was moved to Washington, D. C., in September, 1922, he became Assistant Engineer of the Pittsburgh District of the Portland Cement Association. His work at this time was promotional and educational in the uses of concrete. He answered technical inquiries and expedited field work. His personal interest was in the use of concrete for bridges and roads, and he took great pleasure in inspecting new concrete bridges or stretches of road in the vicinity of Pittsburgh, although inspection was not one of his official duties.

It was with great regret that a second physical breakdown forced him to leave the Portland Cement Association early in 1924. Except for a few months of temporary work at the end of that year, Mr. Somers was never able to work again.

He died of chronic myocarditis, on April 5, 1927, in San Antonio, Tex., where he and Mrs. Somers had gone in October, 1926, hoping that the milder climate would be beneficial to him.

Mr. Somers was a member of the Mt. Lebanon United Presbyterian Church, in Pittsburgh, and of the Theta Xi Fraternity.

In May, 1917, he was married to Rachael Wagner, an Oberlin College classmate, who survives him.

Mr. Somers was elected an Associate Member of the American Society of Civil Engineers on June 6, 1921.

HARRY THOMAS SPENGLER, Assoc. M. Am. Soc. C. E.*

DIED APRIL 23, 1927

Harry Thomas Spengler was born in Bethlehem, Pa., on March 19, 1883. He attended the Bethlehem Public Schools, and when, in 1898, he removed with his parents to Easton, Pa., he enrolled as a pupil there. He prepared for college in the Easton High School and the Bethlehem Preparatory School, and entered Lafayette College in 1905 as a member of the Class of 1909. He was graduated from Lafayette with the degree of Civil Engineer, having been awarded the Bassett Prize in Civil Engineering as first in rank in his class.

Mr. Spengler's first position was with the Maintenance of Way Department of the Lehigh Valley Railroad Company at Easton. In 1911, he became Assistant Engineer in the office of the Chief Engineer, in New York, N. Y., resigning in 1912 to accept appointment to the Faculty of his Alma Mater. His last work with the Lehigh Valley Railroad Company was the supervision of the construction of the Newark Station, at Weequaic Park (Park View Station), N. J.

At Lafayette College he was Instructor in Surveying and Railroad Engineering from 1912 to 1915; from 1915 to 1918, he served as Assistant Professor of Surveying and Railroad Engineering; and from 1918 to 1921 he was Associate Professor of Civil Engineering. In 1920-21 he had charge of the Department of Civil Engineering. In 1921, he was made a full Professor with the title of Professor of Railway Engineering, which position he held until his death.

Professor Spengler had served since 1922 as Engineer for the City of Easton Planning Commission; and was a member of the American Association of University Professors; the Society for the Promotion of Engineering Education; and of the Engineers' Club of the Lehigh Valley.

He was a devoted member of the First Presbyterian Church of Easton and a Trustee of that congregation; a member of Dallas Lodge No. 396, F. and A. M., and of the Northampton County Historical Society, having served as its Secretary for several years. Until shortly before his death, he was a member of the Easton Rotary Club.

While in college he was a member of the Glee Club and in his Senior Year, its leader, as well as a member of the "Calumet Club", "Knights of the Round Table", and the "Sock and Buskin" dramatic organization. He was a member of the Gamma Epsilon Chapter of the Sigma Nu Fraternity, and Alumni Treasurer of the Chapter.

After his graduation from college, Professor Spengler studied music under Professor F. Bristol, of New York. For ten years he was baritone soloist in the choir of the First Presbyterian Church, at Easton, leaving this position to become Choir Director at the First Methodist Episcopal Church, and again

* Memoir prepared by F. O. Dufour, M. Am. Soc. C. E.

leaving this to become Director of the Musical Clubs and of the Chapel Choir at Lafayette College.

Professor Spengler was active in church, college, and civic affairs, and was a leader in all undertakings with which he was connected. His wide acquaintance in the community and his universal tact and good judgment were invaluable to the interests he served.

He was not only a friend of his colleagues on the Lafayette College Faculty, but he was respected and beloved as well by the many students who were privileged to know him as their teacher, adviser, and friend.

He was married in 1910 to Blanche E. Speer, of Easton, by whom he is survived, with two sons, Harry Thomas and Edgar Laubach Spengler, his parents, Mr. and Mrs. A. P. Spengler, a brother, Alfred L. Spengler, of Charleston, W. Va., and a sister, Mrs. John W. Magee, of Germantown, Pa.

Professor Spengler was elected an Associate Member of the American Society of Civil Engineers on January 17, 1921. He was also a member of the Lehigh Valley Section of the Society.